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HYDRO **Newfoundland and Labrador Hydro**

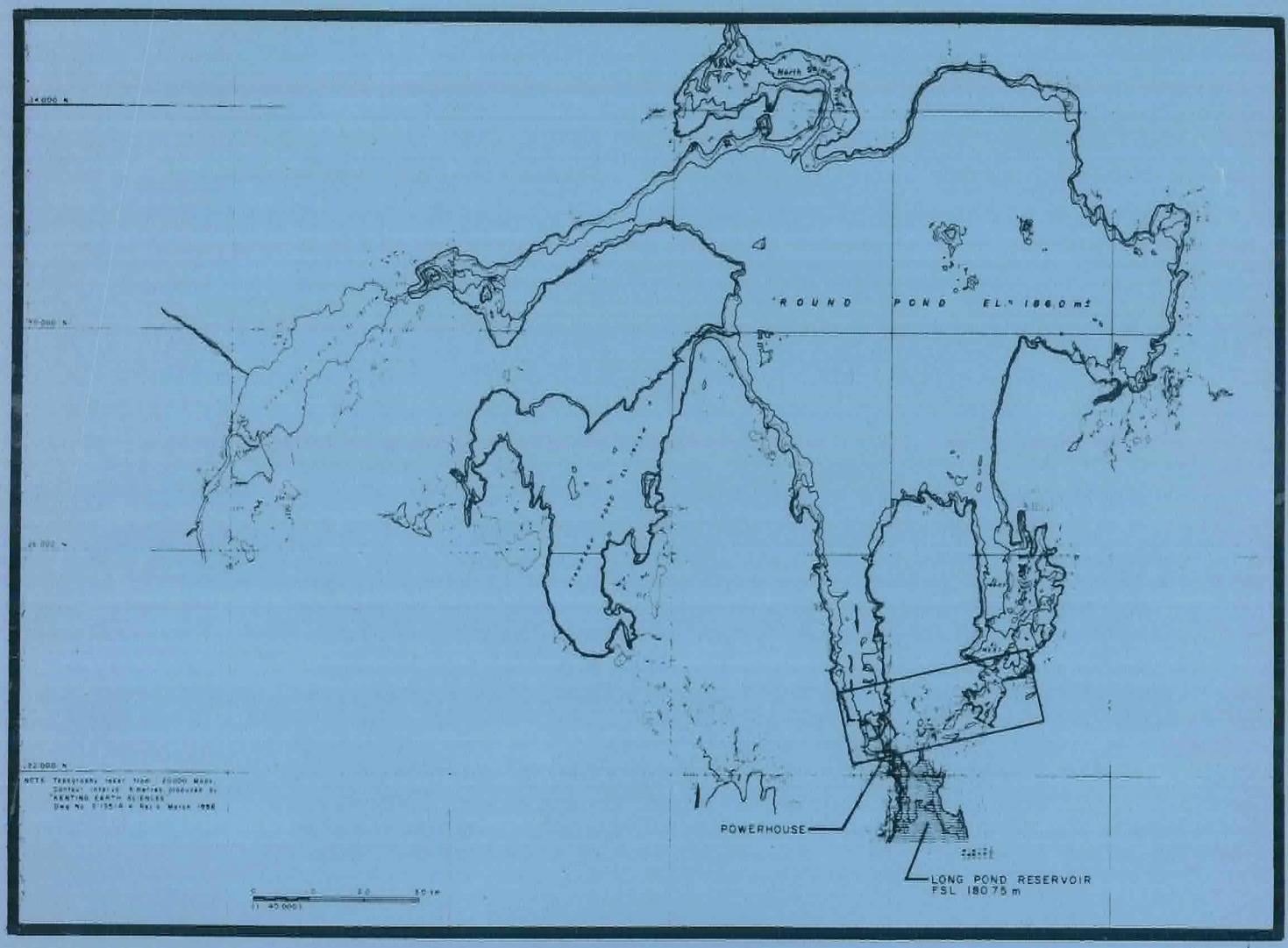
RETURN TO
SYSTEM PLANNING

Feasibility Study

Round Pond Hydroelectric Development

Project no. 30411
September 1988

Volume I Report



SHAWINIGAN/FENCO NEWFOUNDLAND LIMITED

Lavalin

ROUND POND DEVELOPMENT

FEASIBILITY STUDY

VOLUME I

REPORT

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SUMMARY

This Report provides an assessment of the technical feasibility and Capital Cost of the proposed Round Pond Hydroelectric Development situated in the Bay d'Espoir drainage basins. The Report includes geology, hydrology, structure layouts, construction schedule and cost estimates for the complete development. Construction of the proposed project is technically feasible for an installation of one-18 MW unit at a capital cost of $70.0 x 10^6 in July 1988 dollars (excluding IDC, escalation, transmission line, communication and telecontrol facilities) as described herein.

AUTHORIZATION

Formal authorization to proceed with the prefeasibility phase of the Round Pond Hydroelectric Development was received from Newfoundland and Labrador Hydro (HYDRO) by a telex message dated August 6, 1987 following the submission of a Shawinigan/Fenco proposal dated June 12, 1987. After reviewing the preliminary results of the prefeasibility study HYDRO decided to proceed with the feasibility level work. Consequently, an authorization was issued to Shawinigan/Fenco on November 26, 1987 to proceed with the full feasibility study.

INTRODUCTION

The hydroelectric power potential at the outlet of Round Pond into Long Pond Reservoir was identified during a flood study of the Bay d'Espoir drainage area carried out by Acres International Ltd. (Acres). Acres subsequently executed a desk study and produced a short report in November 1985. This preliminary study proposed a development of 10.5 MW under an available generating gross head of 5.8 m between the operating head pond level of Long Pond Reservoir and the tailwater level of the Godaleich Generating Station under normal (average) operating conditions.

Shawinigan/Fenco has upgraded this preliminary study and presented all the findings in this feasibility study report.

GENERAL SCOPE

The study was carried out in accordance with the Terms of Reference outlined in HYDRO's request for proposals dated May 25, 1987.

The study comprised two phases, namely a prefeasibility phase with a limited field program to assess the benefits of the scheme before committing funds either to complete the prefeasibility study report or to undertake a full feasibility phase with a complete field program to confirm the findings of the prefeasibility study and to prepare a final feasibility study report.

The scope of work for each phase is described briefly as follows:

1. Prefeasibility Study:

   - preliminary aerial reconnaissance, subsurface investigations, ground surveys to identify alternative structure layouts.

   - assessment of firm and average energy capabilities of the project.

   - preliminary selection of appropriate generating unit(s).

   - development of a project construction schedule.

   - preparation of cost estimates.

   The above work has been carried out to such an extent as was necessary to provide a capital cost estimate and to determine the energy available in order to allow HYDRO a basis to proceed with further work. The results of the prefeasibility study were presented and discussed with officials of HYDRO in St. John's, Newfoundland on October 5, 1987 in the form of sketches, the results of power and energy studies and a detailed cost estimate. The completion of the prefeasibility study with a prefeasibility report was not requested by HYDRO.

2. Feasibility Study:

- detailed ground surveys and subsurface investigations to verify structure foundation conditions and to confirm sources of construction materials.

- finalize structure layouts.

- finalization of firm and average energy capabilities.

- optimization studies.

- confirmation of electro-mechanical plant and equipment.

- development of a detailed engineering and construction schedule.

- development of a capital cost estimate of the project with monthly cost and cash flow schedule.

The above scope of work was slightly modified by HYDRO to include a supplementary field investigation during the prefeasibility phase; to advance the detailed ground survey work from the feasibility to prefeasibility phase and to delete the survey work to access routes planned for the feasibility phase.

Environmental impact studies and transmission line requirements were excluded from the studies.

This Report presents the results of the Feasibility Study.

SITE INVESTIGATIONS

A program of site investigations was undertaken during the period from August 1987 to June 1988 in support of the technical studies. This program covered the following principal areas:

a) Aerial Photographic Interpretation covering:

- location selection of the main access road.
- identification of construction borrow areas.

b) Topographical survey including:

- centre line profile and cross sections along the structure alignments.
- survey between Long Pond and Godaleich Pond to confirm the available head.

c) Geological investigations including:

- test pitting at several locations along the structure alignments and construction material borrow areas.
- diamond drilling at structure sites and limited diamond drilling in borrow areas.
- material testing.

PROJECT DESCRIPTION AND PARAMETERS

The Round Pond Development is located in the Bay d'Espoir catchment area at the outlet of Round Pond. At this location a natural head of approximately 12 metres is available from a series of rapids between Long Pond Reservoir and Round Pond and between Round Pond and Godaleich Pond. Water for power generation will come from the regulated discharge of the existing generating station at Godaleich Pond together with the natural inflows into the Round Pond drainage basin.

Although a natural head of about 12 metres is available, the gross head for hydroelectric development at Round Pond has been restricted to 11.3 metres in order to avoid affecting the existing level of Godaleich Pond.

The Round Pond Development, excluding transmission line, communication and control facilities, will comprise the following:

- permanent access road,
- main earthfill dam,
- saddle dams,
- gated spillway with low ogee crest,
- power intake canal,
- intake/surface powerhouse/tailrace,
- Pit turbine 18 MW single unit,
- switchyard,
- fish passage facility.

The principal parameters for the Round Pond Development are listed in the following table.

The general layout of the proposed Development is shown on Drawing 7.1.

PRINCIPAL PARAMETERS OF ROUND POND DEVELOPMENT		
DESCRIPTION	UNIT	QUANTITY
GENERAL		
Total Catchment Area	km^2	950
Average Flow	m^3/s	163.3
Maximum Plant Flow	m^3/s	193.1
Firm Flow	m^3/s	144.3
Average Plant Flow	m^3/s	161.8
Gross Head	m	11.3
Net Head	m	10.8
Full Supply Level (FSL)	m	192.0
Maximum Tailwater Level	m	184.20
Normal Tailwater Level (Long Pond Reservoir)	m	180.75
Maximum Routed Flood Inflow	m^3/s	3500
Spillway Capacity at MFL	m^3/s	2790
Design Flood during Construction (1 in 20 year)	m^3/s	500
Maximum Flood Level (MFL)	m	195.0
POWER AND ENERGY		
Installed Capacity	MW	18
Number and Type of Units	No.	1/Pit
Turbine Rated Output	MW	19.1
Average Annual Energy (Round Pond)	GWh	132
Firm Annual Energy (Round Pond)	GWh	109
STRUCTURES		
Permanent Access Road	km	12
Earthfill Dam		
- Crest Elevation	m	196.5
- Crest Width	m	6.0
- Length	m	800
Saddle Dam		
- Crest Elevation	m	196.5
- Crest Width	m	5.0
- Length	m	1050
Spillway Dam		
- Ogee Crest Elevation	m	185.0
- Ogee Crest Width	m	11
- Spillway Bays	No.	4
- Spillway Gates (Vertical Lift Type)	m x m	11 wide x 7.5 high
Power Canal		
- Length	m	290
- Bed Width	m	22
- Invert Elevation	m	179.5
Powerhouse (Single Pit Turbine)		
- Length	m	48.0
- Width	m	12.5
Fish Passage Facility		
- Length	m	80 m
- Width	m	1.5 m
- Assumed flow for the months of Aug, Sept, Oct and Nov	m^3/s	2.0

CAPITAL COST ESTIMATES

The total project costs in July 1988 Dollar terms are as follows:

	Description	Cost $ x 1,000
1.	Civil Works	33,669
2.	Electrical and Mechanical Equipment	20,007
	Construction Costs - Sub-Total	53,676
3.	Engineering and Construction Management	6,446
4.	Owner's Cost	3,377
5.	Contingencies	6,373
	Capital Costs (July 1988) TOTAL	69,872
6.	Escalation	7,536
7.	Interest During Construction	11,428
	TOTAL PROJECT COST	88,836

The above estimated cost of Civil Works include costs for the access road which was provided by HYDRO. The costs of telecontrol equipment and transmission lines (beyond the switchyard) are not included in these estimates.

In preparing the estimates for the project, emphasis has been placed on evolving conservative but realistic prices which recognize all conditions that apply to the project and its location and assume an element of competition within the construction market.

POWER AND ENERGY

The Round Pond Development will have the following capacity and energy capabilities:

Nominal Installation	18 MW
Firm Energy	109 GWh per annum
Total Average Energy	132 GWh per annum
Average Capacity Factor	84%

IMPLEMENTATION SCHEDULE

The implementation schedule is based on the assumption that the engineering design could start in April 1, Year 1 and the project release date is July 1, Year 1. The implementation schedule is then proposed with the following key dates:

HYDRO Approval to proceed with Engineering	- April 1,	Year 1
Project Release Date	- July 1,	Year 1
Award of Permanent Access Road Contract	- July 1,	Year 1
Approval and issue of Tenders for Main Civil Works Contract	- July 1,	Year 1
Award of Main Civil Works Contract	- October 1,	Year 1
Approval and issue of Tenders for Hydraulic Gates Contract	- September 15,	Year 1
Award of Hydraulic Gates Contract	- December 1,	Year 1
Approval and issue of Tenders for Generating Plant Contract	- August 1,	Year 1
Award of Generating Plant Contract	- February 1,	Year 2
Approval and issue of Tenders for Miscellaneous Electrical and Mechanical Equipment Contract	- June 1,	Year 2
Award of Miscellaneous Electrical and Mechanical Equipment Contract	- October 1,	Year 2
Commencement of Reservoir Impounding	- July 1,	Year 3
Works Ready to Admit Water	- October 1,	Year 3
On Power	- December 15,	Year 3

PROJECT EVALUATION

The optimum installation for the Round Pond Development has been evaluated in terms of net benefits calculated from total system operating costs with and without the proposed development. The optimum installation for the Round Pond Development was found to be 18 MW and the net gain in average system energy 132 GWh.

The optimization was based on a value of energy of $0.76/KWh in 1987 dollars.

ENVIRONMENTAL CONSIDERATIONS

This is not within the scope of the Feasibility Study.

TRANSMISSION FACILITIES

This is not within the scope of the Feasibility Study.

1. INTRODUCTION

1. INTRODUCTION

1.1 GENERAL

This introductory section describes the background to the feasibility study of the Round Pond Hydroelectric Development in Newfoundland and includes the scope of work covered by the study program.

1.2 BACKGROUND AND PREVIOUS ENGINEERING STUDIES

The hydroelectric power potential at the outlet of Round Pond was identified during a flood study of the Bay d'Espoir drainage area by Acres International Ltd. A desk study was subsequently carried out by Acres and a short report produced in November 1985. The layout proposed in this study featured two concrete gravity overflow spillways in each of the two channels at the Round Pond outlet connected by earthfill dykes. A large section of these dykes was designed as a fuse plug type spillway to fail during the Probable Maximum Flood in order to maintain the lake at its natural level. A combined intake/powerhouse, housing a single 10.5 MW Pit turbine, was located on the left bank of the east channel and on the north bank of the entrance to Long Pond Reservoir. Water was conveyed to the powerhouse by a 300 metre long power canal with an uncontrolled intake on this same left bank.

Shawinigan/Fenco were retained by Newfoundland and Labradro Hydro (HYDRO) to carry out a prefeasibility study to upgrade the preliminary study by Acres. Findings of the prefeasibility study are provided in Appendix B in this report.

1.3 AUTHORIZATION FOR REPORT

Formal authorization to proceed with the prefeasibility phase of the Round Pond Hydroelectric Development was received from

HYDRO by a telex dated August 1987 following the submission of a Shawinigan/Fenco proposal dated June 12, 1987.

Following a presentation of the results of the prefeasibility study, HYDRO gave authorization to Shawinigan/Fenco on November 26, 1987 to proceed with the full feasibility Study.

1.4 SCOPE OF WORK

1.4.1 General

The study was carried out basically in accordance with the Terms of Reference outlined in HYDRO's request for proposals dated, May 25, 1987 but with some changes described in Section 1.4.3.

1.4.2 Original Scope

The study comprised two phases; a prefeasibility phase with a limited field program to assess the benefits of the scheme, before making a monetary commitment for further study; and a feasibility stage with a full field program including diamond drilling to confirm the findings of the prefeasibility study. The scope of work for each phase is described briefly as follows:

1) Prefeasibility Study

   a) Aerial photo interpretation of the project area to assess borrow areas for construction materials, project geology and alternative routes for the access road.

   b) Geotechnical field investigations, comprising aerial and ground reconnaissance of reservoir area, structure sites

and access routes and test pitting in these areas to confirm the aerial photo interpretation.

c) A limited survey program to confirm the available head and collect information to enable preliminary structure layouts and an estimate of quantities to be prepared.

d) Developing and costing alternative structure layouts to allow selection of the best scheme for the Development.

e) Firm and average energy capabilities of the project using flow data from previous comprehensive studies of the Bay d'Espoir system.

f) Selection of unit type and assessment of single unit schemes against multi-unit schemes.

g) Development of construction schedule.

h) Preliminary Capital cost estimate of the project along with cost and cash flow schedules.

Following completion of the above work, the results were to be reviewed with Newfoundland and Labrador Hydro, to enable a decision to be made on further commitment to the Development.

2) Feasibility Study

a) Geotechnical investigation comprising test pitting and limited diamond drilling in borrow areas to determine quality and quantity of construction materials, diamond drilling at structure sites to determine depth of

overburden and quality of bedrock and laboratory testing of construction materials.

b) A field survey program to obtain topography of all structure sites (centre line and typical sections).

c) Hydrological studies of the watershed to develop firm and average energy capabilities, design flood magnitude and construction flood magnitude.

d) Firm and average energy capability of the site using the regulation model developed for Newfoundland and Labrador Hydro for the Bay d'Espoir System.

e) Selection of the access road route using mapping and aerial photo interpretation work and ground reconnaissance.

f) Survey of the major stream crossings (centre line profile and cross sections).

g) Confirmation of selected layout, unit selection and number of units.

h) Development of detailed construction schedule and a monthly project schedule.

i) Capital cost estimate of the project including quantities for each facet of construction, monthly cost and cash flow schedules.

Environmental impact studies and transmission line requirements were not part of the scope of this study.

1.4.3 Changes to Scope of Work

As a result of a ground survey carried out as part of the field investigation for the prefeasibility study, the value of the gross head available between the operating head pond level of Long Pond and the tailwater level of the Upper Salmon Generating Station was found to be approximately 12 m substantially different from the 5.8 m quoted in the preliminary report by Acres. This discrepancy was reported to be attributed to the use of different datum levels for construction of the Bay d'Espoir and Upper Salmon Developments.

Because of this increase in head the following work was required in addition to that described in Section 1.4.2:

i) addition of a long saddle dam in a low lying area some 1 kilometer east of the site, required that a supplementary field program be carried out to determine geotechnical and topographical information at alternative axes to enable a more reliable quantity take-off to be made.

ii) investigation of alternative structure alignments which were not feasible at the lower head.

iii) optimization of the FSL.

HYDRO transferred the detailed survey work from the feasibility phase of the study to the prefeasibility phase and deleted the survey work for the access road described in items 2(e) and 2(f).

1.5 ACKNOWLEDGEMENTS

Shawinigan Engineering Company Limited and Fenco Newfoundland Limited wish to acknowledge the cooperation and assistance of the staff of Newfoundland and Labrador Hydro in carrying out this study.

Thanks are also extended to others who have contributed to assistance and information amongst whom are:

Golder Associates	- Geotechnical Field Investigation
J.P. Bouzane and Associates	- Aerial Photo Interpretation.
Allterra Drilling Inc.	- Drilling Contractor

2. DESCRIPTION OF SITE

2. DESCRIPTION OF SITE

2.1 LOCATION

The Round Pond Development is located in the Bay d'Espoir catchment between Long Pond Reservoir and Godaleich Pond. The general location of the proposed development is shown on Dwg. 2.1.

The Bay d'Espoir Development is also illustrated on Dwg. 2.1 and comprises a number of dams and diversion canals which divert run-off from the Victoria River, White Bear River and Grey River catchments into the Salmon River. The latter in turn was dammed to create the Long Pond Reservoir, which supplies water to two power plants at the head of Bay d'Espoir. These operate under a gross head of 181 m and have a total installed capacity of 600 MW.

Water from the Victoria River diversion is regulated from Victoria Reservoir and water from the White Bear River diversion is regulated from Meelpaeg Reservoir which was created by diverting the Grey River.

The Upper Salmon Development, which is the most recent of the Bay d'Espoir developments is situated between Great Burnt Lake, and Round Pond immediately upstream of Long Pond Reservoir. Flows from the North Salmon River were diverted by a dam at the outlet of Great Burnt Lake where a headpond was created through a series of ponds extending from Crooked Lake through Great Burnt Lake to Cold Spring Pond. The Upper Salmon powerhouse which is located near Godaleich Pond, develops approximately 52 m of the approximate total head of some 87 m between Meelpaeg Reservoir and Long Pond Reservoir and has an installed capacity of 84 MW.

A natural head of approximately 12 m is available at the Round Pond outlet from a series of rapids between Long Pond and Round Pond and between Round Pond and Godaleich Pond. However to avoid

affecting the existing water levels of Godaleich Pond, the gross head for the hydroelectric development at Round Pond has been restricted to 11.3 m. This is discussed in detail in Section 6.0.

2.2 SITE DESCRIPTION

The Development is located at the outlet of Round Pond. The outlet comprises two channels; the main channel is located at the west side of the outlet and a tributary channel at the east side.

The west channel is bounded by a relatively high right abutment and a lower left abutment and comprises a series of rapids some 700 m long. The east channel is relatively flat upstream from a short section of rapids which control the outflow into the main channel. The island created by these channels upstream of their confluence is approximately 400 m long and 250 m wide and is partly submerged during normal flood conditions and fully submerged during the peak flood.

A topographic saddle exists on the left abutment of the main channel downstream of the confluence, after which contours start to rise. A low lying area is located some 1 km to the east of the site just south of the easterly tip of Round Pond.

2.3 ACCESS TO THE SITE

Presently access to the site is possible only by float plane, helicopter, or by boat from Long Pond Reservoir.

The nearest point of vehicular access lies some 12 km to the west of the site on the Upper Salmon Road, which was constructed as the main access to the Upper Salmon Development. Several routes to link the Round Pond site to the Upper Salmon Road were investigated by HYDRO.

2.4 GENERAL DESCRIPTION OF THE DEVELOPMENT

The Round Pond Development is illustrated on Dwg. 7.1 and comprises construction of the following:

- a gated spillway in the east channel at the outlet to Round Pond, having the capability of discharging the project design flood of 2790 m^3/s at the maximum flood level of El. 195.0.

- an earthfill dam (West Causeway, Center Causeway and East Causeway) 800 m long and 10 m maximum height extending from the right bank of the west channel to the left bank of the east channel;

- two small saddle dams (Sadde Dam No. 1 and No. 2) approximately 2 m high on the left bank of the east channel;

- a saddle dam (Saddle Dam No. 3) 1050 m long and 7.5 m maximum height located approximately 1 km to the east of the project area just south of the easterly tip of Round Pond;

- an intake canal 300 m long leading to a combined intake/powerhouse equipped with a single 18.0 MW Pit turbine, operating under a net head of 10.8 m;

- a short tailrace channel connecting the powerhouse with Long Pond Reservoir;

- a fish passage 80 m long, connecting the intake channel just upstream of the powerhouse with the Long Pond Reservoir.

The principal parameters for the development are summarized in Table 2.1.

TABLE 2.1

PRINCIPAL PARAMETERS OF ROUND POND DEVELOPMENT		
DESCRIPTION	UNIT	QUANTITY
GENERAL		
Total Catchment Area	km^2	950
Average Flow	m^3/s	163.3
Maximum Plant Flow	m^3/s	193.1
Firm Flow	m^3/s	144.3
Average Plant Flow	m^3/s	161.8
Gross Head	m	11.3
Net Head	m	10.8
Full Supply Level (FSL)	m	192.0
Maximum Tailwater Level	m	184.20
Normal Tailwater Level (Long Pond Reservoir)	m	180.75
Maximum Routed Flood Inflow	m^3/s	3500
Spillway Capacity at MFL	m^3/s	2790
Design Flood during Construction (1 in 20 year)	m^3/s	500
Maximum Flood Level (MFL)	m	195.0
POWER AND ENERGY		
Installed Capacity	MW	18
Number and Type of Units	No.	1/Pit
Turbine Rated Output	MW	19.1
Average Annual Energy (Round Pond)	GWh	132
Firm Annual Energy (Round Pond)	GWh	109
STRUCTURES		
Permanent Access Road	km	12
Earthfill Dam		
- Crest Elevation	m	196.5
- Crest Width	m	6.0
- Length	m	800
Saddle Dam		
- Crest Elevation	m	196.5
- Crest Width	m	5.0
- Length	m	1050
Spillway Dam		
- Ogee Crest Elevation	m	185.0
- Ogee Crest Width	m	11
- Spillway Bays	No.	4
- Spillway Gates (Vertical Lift Type)	m x m	11 wide x 7.5 high
Power Canal		
- Length	m	290
- Bed Width	m	22
- Invert Elevation	m	179.5
Powerhouse (Single Pit Turbine)		
- Length	m	48.0
- Width	m	12.5
Fish Passage Facility		
- Length	m	80 m
- Width	m	1.5 m
- Assumed flow for the months of Aug, Sept, Oct and Nov	m^3/s	2.0

3. FIELD INVESTIGATION PROGRAM

3. FIELD INVESTIGATION PROGRAM

3.1 GENERAL

The primary objective of the field investigation program for the study was to obtain detailed surface and sub-surface data at the site and to incorporate such data into preliminary designs of the various components of the proposed development to the extent necessary to prepare a realistic capital cost estimate. This cost estimate is to be at a level of detail that will allow adequate assessment of the technical feasibility and total Capital Cost of the development.

Basically, the field investigations have included field reconnaissance study, topographical surveys, geological mapping, ground reconnaissance and subsurface investigations. The field work has been supplemented by laboratory tests on soil and rock samples.

Two separate field programs were originally planned for the study as follows:

1) a preliminary program to be carried out at the prefeasibility stage comprising test pitting at the structure sites and borrow areas to confirm aerial photo interpretation work; ground and aerial reconnaissance of the access road; and preliminary survey work.

2) a detailed site investigation at the feasibility phase comprising diamond drilling and test pitting along the structure alignments and the borrow areas, survey of major stream crossings along the access road and a centre line survey (including cross sections) at the structure sites.

As previously mentioned in Section 2 the detailed survey work was advanced from the feasibility program to the prefeasibility phase. As part of this work a survey was carried out between the operating head pond level of Long Pond and the tailwater level of the Upper Salmon Generating Station to confirm the available head between these two points. This survey found the available head at the Round Pond site to be approximately 12 m, substantially different from the 5.8 m quoted in the preliminary report by Acres. This discrepancy was found to be attributed to different datum levels used during construction of the Bay d'Espoir and Upper Salmon Developments. The correction to be applied to the elevations at the Upper Salmon Generating Station to correspond with a Long Pond Datum was computed as +5.483 m.

As a result of this increase in head, a long saddle dam is now necessary in the low lying area some 1 kilometer east of the site in order to effect closure at Round Pond. Only 1:50,000 scale mapping was available for this area and from aerial photography it was evident that bog areas were extensive. At the conclusion of the prefeasibility study, it was recommended that the saddle dam area should be surveyed and an investigation made of the subsurface conditions to enable a more accurate cost estimate to be made. The field program for the feasibility study had to be rescheduled to the summer of 1988 to avoid winter drilling. A supplementary site investigation program was subsequently undertaken for the saddle dam area from November 25 to December 3, 1987.

The detailed field investigation was subsequently carried out from May 16 to June 8, 1988.

A summary of the field programs and the material testing carried out on the samples collected is given in the following sections. Full details are provided in Volume II.

3.2 FIELD INVESTIGATION PROGRAM - PREFEASIBILITY STUDY

3.2.1 Aerial Photo Interpretation

Prior to launching the field program a surficial geology map was prepared from aerial photo interpretation. This map covered the structures area (with the exception of the saddle dam) and one of the alternative access routes which ran in a generally westerly direction from the site. This map is shown on Dwg. 3.1.

3.2.2 Geotechnical

3.2.2.1 Access Road

A ground reconnaissance of the eastern end of the access route was made. Several areas along the route were probed to verify the surficial characteristics identified from the aerial photo interpretation. Aerial reconnaissance using a helicopter was carried out on the remainder of the route. Selected areas were probed and samples collected from the potential borrow pit locations along the route which had been identified from the aerial photography.

The surficial cover in the glacial till areas was found to be in the order of 0.25 m. Probing in the peat areas typically gave thicknesses less than 0.5 m.

Probing in the borrow areas indicated that surficial cover over the underlying silty sand and/or sand till ranged from 0.3 m to 0.5m.

3.2.2.2 Powerhouse and Power Canal

A total of five test pits (Nos. 1-5) were dug. The positions of these are indicated on Dwg. 3.2. The test pits were excavated to fairly shallow depths (max. depth 1.5 m) because of the presence of dense cobble and boulder layers, which made hand excavation difficult.

3.2.2.3 Causeway

One test pit was excavated approximately midway along the causeway alignment. Excavation beyond a depth of 0.6 m was impossible. This was assumed to be because of bedrock since joint measurements taken, matched those of the exposed bedrock in the east channel.

3.2.3.4 Topographic Survey

A ground profile was surveyed along the centreline of the power canal, powerhouse site and closure dyke and cross sections taken at selected locations. River bed profiles were taken approximately at the control sections of the rapids and water elevations and bed elevations at key locations along the rapids on the west channel. Permanent monuments were established at the structure sites as indicated on Dwg. 3.2.

Since there was no geodetic datum established at the site, all levels were referred to the elevation of Long Pond Reservoir which is recorded daily. The water level used was Elevation 179.69 m as recorded on August 27, 1987.

To confirm the available head between Long Pond Reservoir and Godaleich Pond, a level run was carried out between the tailrace of the Upper Salmon Generating Station and Round Pond. The water level in Round Pond was assumed to be constant from a point approximately 1 km upstream of the Round Pond outlet to a point some 800 m from the outfall of the West Salmon River and therefore no survey was done between these two points. The difference in water levels between Round Pond and Godaleich Pond and Round Pond and Long Pond were found to be 6.854 m and 6.129 m respectively giving a total available head of 12.983 m.

3.3 SUPPLEMENTARY FIELD PROGRAM - SADDLE DAM AREA

3.3.1 Geotechnical

The area investigated lies some 1 kilometer east of the outlet of Round Pond. Two alternative axes, identified as Alignment A and Alignment B on Dwg. 3.2, were investigated. The extent of the investigation is summarized as follows:

1) Alignment A

A series of peat probes were carried out along the central and eastern sections of the dam alignment as indicated on Dwg. 3.2. Thicknesses of peat ranged from 0.3 m to 2.8 m with the greater thickness encountered over the eastern section. Typically a layer of cobbles and boulders was encountered under the peat. No test pits were dug at this alignment.

2) Alignment B

Peat probing was carried out as indicated on Dwg. 3.2. The depth of peat encountered ranged from less than 0.3 m to a maximum of 1.0 m. The deepest area of peat was encountered at the centre of the small fen at the extreme end of the alignment. The peat was underlain by cobbles and boulders.

Four test pits were excavated by hand in the peat areas and generally 0.3 to 0.5 m of peat was encountered overlying cobbles and boulders.

Three test pits were excavated by hand in areas of till deposits. Test pits were typically one metre deep, generally revealing a 0.1 m thick root mat underlain by a layer of silty sand with some gravel, cobbles and boulders which are generally classified as glacial till.

3.3.2 Topographic Survey

i) Available Mapping

The 1:2500 topographic mapping of the Round Pond outlet was extended to cover the saddle dam area. This was done by photogrammetric restitution, using the lake elevations for control as no surveyed ground control points existed for this area.

ii) Ground Survey

The following survey work was carried out:

1) Extension to cross sections surveyed in the power canal, powerhouse area and the causeway during the prefeasibility field program.

2) Additional cross sections were surveyed along the causeway alignment on the west side of Round Pond. This was required because of the additional head at the site.

3) Vertical control was transferred to Saddle Dam Alignment B.

4) A profile and cross sections were taken along saddle dam Alignment B.

5) A bench mark was transferred from Godaleich Pond to the Upper Salmon Generating Station.

No survey work was done at Saddle Dam Alignment A. However, the 1:2500 topographic map prepared for the saddle area agreed well with the ground survey for Alignment B and was used for comparing quantities between the two axes.

3.3.3 Aerial Photo Interpretation and Terrain Analysis

An aerial reconnaissance of the proposed access routes was carried out with representatives of HYDRO to confirm the surficial characteristics identified from the aerial photo interpretation. No test pits were excavated, at this time. An aerial reconnaissance of the proposed flooded area was also made and areas of blowdown and types of timber noted, for subsequent use in preparation of the reservoir clearing map shown on Dwg. 9.1.

3.4 FIELD INVESTIGATION PROGRAM - FEASIBILITY STUDY

3.4.1 Topographic Survey

The following survey work was carried out:

1) Centre line profile for Saddle Dam Alignment A.

2) Horizontal control transferred from the structures area to the saddle dam area.

3) Setting out and establishing ground levels at the borehole positions indicated on Dwg. 3.2.

3.4.2 Geotechnical

3.4.2.1 Diamond Drilling

The diamond drilling program included a total of 6 boreholes along the main structure alignments, 1 hole in Saddle Dam Alignment A, at the deepest section of bog, 2 holes in the impervious borrow areas and one hole in the quarry area near the centre of the

island. The borehole layout is given on Dwg. 3.2. This amounted to a total of 96 m of drilling over a period of 3 weeks.

The type of drill used was a Boyles BBS-15 and the core size was BQ (37.5 mm O.D.).

Details of borehole logs are presented in Volume II.

3.4.2.2 Test Pits and Peat Probing

Test pits were dug in the esker deposits located to the south of the site with a backhoe equipped for operation in forested areas. The test pits were typically 2.5 m deep. Test pits were excavated by hand in borrow area T-3 adjacent to the site to a depth of 2.2 m.

Peat probing was carried out on the higher sections of Alignment A and on locally higher ground near the deep section of bog in the area of Borehole No. 9.

The locations of all test pits and peat probings are indicated on Dwg. 3.2.

3.5 MATERIAL TESTING

3.5.1 Field Testing

The following field tests were carried out during the course of the investigations:

- Standard Penetration Test to provide SPT N index value for estimating soil density and soil classification.

- Packer Test, carried out during drilling to assess bedrock permeabilitiy.

- Groundwater Measurements to provide an estimate of the groundwater table.

3.5.2 Labratory Testing

The following tests were carried out in Golder's laboratory on selected samples collected during the field investigations:

- water content on most soil samples.

- grain size analysis by sieving and hydrometer.

- Atterberg Limits to determine the liquid and plastic limits of the soil.

- unconfined uniaxial compression on five selected rock core samples.

Full details of the test results are given in Volume II.

4. GEOLOGY

4. GEOLOGY

4.1 REGIONAL GEOLOGY

The geology of the Round Pond area is dominated by sedimentary and volcanic rocks of the Middle Ordovician, Bay d'Espoir Group and later intrusive rocks. Most sedimentary rocks are turbidites and the volcanic rocks are felsic and pyroclastic.

Intrusive rocks in this area are generally composed of biotite and biotite muscovite granodiorite. The Devonian-aged North Bay Granite outcrops to the south of the proposed site.

The Baie d'Espoir Group was divided into four formations by Colemen Sadd (ref. 1). These formations are referred to as the Salmon River Dam, Riches Island, St. Joseph's Cove and North Steady Pond Formations.

The Salmon River Dam formation clearly dominates in the site geology. This formation occurs in contact with the North Bay Granite. It is composed of purplish gray, weathered to brown and calc-silicate beds and is considered by Coleman Sadd to demonstrate remarkable regional consistency of the fine grain sizes of its constituents.

The Salmon River Dam Formation is considered to be partly older than the St. Joseph's Cove and North Steady Pond Formations. It outcrops where the sediments have been domed upwards by granitoid intrusions. The Riches Island Formation appears to have a lateral interfingering relationship with the St. Joseph's Cove Formation. Also the St. Joseph's Cove and North Steady Pond Formations have a gradational contact; they both overlie the Salmon River Dam Formation and are assumed to be laterally equivalent.

All formations of the Baie d'Espoir Group show evidence of two deformations. The initial deformation formed isoclinal folds throughout the group. There is evidence that many of the rocks were unlithified at the onset of this event. This deformation is illustrated by folds and crenelation cleavage developed to the southeast of the study area and is evident in sections of core recovered.

Low pressure, regional greenschist metamorphism has affected the Baie d'Espoir Group. Adjacent to the granodiorite and tonalite instrusions the metamorphic grade is in the amphibolite facies.

4.2 SITE GEOLOGY

The Development is located in the south central area of the island portion of Newfoundland, north of Baie d'Espoir. Most of the study area forms a peneplain with an elevation of 150 to 300 m, which probably corresponds to the Lawrence level of erosion according to Twenhofel and MacClintock (Ref. 2).

Pleistocene glaciation has spread a thick mantle of lineated, eroded and dissected morainal deposits over much of the area. These are generally unsorted, structureless materials such as till derived through glaciation and usually contain material with size ranges from boulder down to clay sizes. The till mantle thickness in the area ranges from less than 1 m to greater than 10 m.

Isolated boulder trains occur in several areas, generally adjacent to ridges. These boulders are generally greater than 1 m in size and are typically limited in aerial extent. Most boulder sized material observed was of granitoid composition.

These ridge moraine-type features generally run north-south with peatland deposits running in a similar orientation between them.

The findings from the field investigations carried out for these studies indicate that the till in this area is somewhat stratified and of variable composition. Tills in the borrow search area along the proposed access route are dominated by sand, silt and gravel sized fractions with generally a minor proportion of cobbles and boulders.

The area to the east of the site is dominated by a relatively large peatland deposit. The bog in this area is believed to be typically less than 3 m in thickness and underlain by bouldery till deposits.

Two eskers run approximately north-south through the area being considered for the access road (Ref. Dwg. 4.1). These linear features have an abundance of near surface coarse material. However the core of these eskers are reported to consist primarily of sand sized material.

4.3 BORROW AREAS

The locations of the proposed borrow areas for the construction materials are identified on Dwg. 4.1.

Borrow areas T-1, T-2 and T-3 are located adjacent to the site and will be used for glacial till for the causeway and saddle dam construction. The deposits are deep with suitable material (containing from 20-45% fines) lying some 1-2 metres below ground level. At the time of the investigations the groundwater level in these areas was found to be high and it is envisaged that extraction of the material will be restricted to the deposits lying above the water table to avoid the need for stockpiling and drying before placing. Because of the high silt content, these on-site borrow materials will become unworkable when wet.

Borrow areas S/G 1 to S/G 10 are located in esker deposits which extend from a point some 1.5 km south of the site to the Upper

Salmon Road. There is an abundance of sand sized material in these deposits which can be used, after processing, for filters in the embankments and fine aggregate for concrete production. Much of the esker borrow material has a high silt content (in the 40% range) and thus considerable washing and screening will be needed to produce usable clean aggregate. Coarser aggregate for concrete production is available from SG/9. However it appears from the site investigations that the deposit may only have a thin veneer of this coarser material, in which case there would be an insufficient quantity to meet the total construction requirements. The balance of the coarse aggregate would be supplied from crushed rock from the excavation for structures.

4.4 FOUNDATION CONDITIONS

4.4.1 Powerhouse and Power Canal

The powerhouse will be founded on a competent fine to medium grained sandstone. However in the area of the powerhouse and power canal there is an extensive stratum of fine grained silty soils in conjunction with a relatively high water table. The depth of this deposit is in the order of 9 m. Because of this high water table, careful in-excavation groundwater control will be required to achieve excavation slopes of 2:1.

4.4.2 Spillway

The spillway will be founded on competent sandstone. Permeability tests carried out during the drilling program indicate that the jointing in the rock is tight and hence only slush grouting is envisaged as a foundation treatment prior to concreting.

4.4.3 Causeways

The results of the material testing on samples collected during the

site investigations indicate that some of the overburden covering the site is relatively course granular till and would be permeable and could be subject to piping. For this reason the cut-offs for the Centre Causeway and West Causeway have been assumed to extend to bedrock, which lies some 1 to 2 m below ground level. For the greater part of the alignment along the West and Centre Causeways the indications are that the rock has a low permeability and therefore only slush grouting will be required before placing the till. However, possible fractured rock zones are indicated from packer tests carried out at borehole Nos. 7 & 8 on areas of the West Causeway. These areas may require pressure grouting or other foundation treatment.

Because of the extensive overburden depth along the alignment of the East Causeway, the core trench will extend to suitable till material.

4.4.4 Saddle Dams

As indicated by borehole No. 9, the overburden cover along the alignment of saddle dam No. 3 is deep. The cut off trench will therefore extend to suitable till material.

5. HYDROLOGY

5. HYDROLOGY

5.1 INTRODUCTION

Detailed hydrological analysis was not necessary for the current studies as previous work was used for flood and energy estimates. Hydrological input for the prefeasibility study was provided by HYDRO. This input consisted of average monthly inflows for all the major sub-basins of the Bay d'Espoir system.

A regulation study (the Bay d'Espoir Regulation Study) was completed by Acres in late 1987. The model was further developed, using the latest system information and an expanded flow base of 37 years (1950 to 1986), and was calibrated against the existing Bay d'Espoir system. The model has the capability to determine the effects of adding new developments, such as Round Pond, to the total system. Using this model, flood routing and energy generation estimates for the Round Pond Feasibility Study were subsequently computed by Newfoundland and Labrador Hydro, from hydraulic design input data developed from the feasibility study.

5.2 DRAINAGE AREAS

The drainage basins which comprise the Bay d'Espoir System are illustrated on Dwg. 2.1. The Round Pond Development forms part of the Salmon River Drainage Basin and has an incremental drainage area of 950 km^2. After developing the Round Pond site, the net drainage area of the Bay d'Espoir System will not be changed. A review of the local Round Pond catchment area shows that it may be possible to divert part of the D'Espoir Lake and Ahwachanjeesh Pond watersheds into the Round Pond Development, in which case the net increase in drainage area to the total system would be 236 Km^2. These diversions were however studied separately by HYDRO and were found to be marginal and were not considered in this study.

5.3 FLOW DATA

As part of the Bay d'Espoir Regulation Study, 37 years of monthly stream flows (1950-1986) were reconstituted for the Bay d'Espoir drainage system from recorded lake and reservoir levels and records of operations at the various plants and reservoirs within the system. Incremental inflows to the subcatchments were prorated by drainage area. These sequences of reconstituted streamflows are tabulated in Volume III and were used in the Bay d'Espoir Regulation Model to generate the energy estimates for the optimization studies, described later in Section 6.0.

5.4 PROJECT DESIGN FLOOD

In view of the findings of the Bay d'Espoir Flood Analysis and Alternatives Study completed in 1985, it was decided by HYDRO that the design flood for Round Pond should be the PMF. Since the flows passing through Round Pond are regulated by large reservoirs and lakes located upstream, the PMF for the Round Pond development would depend on the operational procedure of the entire Bay d'Espoir System, particularly on the upstream components. The following procedure was adopted for the derivation of the PMF hydrographs in the 1985 Flood Study:

- the Probable Maximum Precipitation (PMP) centred in the Round Pond sub-catchment was derived with a spatial distribution to the remaining sub-catchments,
- unit hydrographs for each sub-catchment were estimated,
- using the derived unit hydrographs, the inflow hydrographs corresponding to the PMP proportions of each sub-catchment were calculated.

The Probable Maximum Flood (PMF) hydrographs as developed were used in the present study as design flood inputs to the Bay d'Espoir Regulation Model for the determination of the spillway capacity and are presented in Table 5.1.

At Victoria Lake and Meelpaeg reservoir, the starting levels were set according to the flood rule curves such that the maximum flood would be handled mainly by the available storage. At other locations in the system, adequate spillway capacities were assumed to exist and the FSL to be the initial reservoir levels. At Round Pond, the starting level was taken as 192.0 m (FSL) and the maximum allowable surcharge was set at 3 m to give a Maximum Flood Level (MFL) of 195.0 m. The PMF hydrographs were routed through the Bay d'Espoir System with 6-hour intervals. The peak inflow to Round Pond reservoir was computed to be 3500 m^3/s and the peak outflow was 2790 m^3/s at an elevation of 194.9 m. The inflow and the routed outflow hydrographs at Round Pond are shown in Figure 5.1.

The preliminary discharge rating curve for the spillway used in the routing is given in Fig. 5.2. Full details of the computer output for the flood routing are given in Volume III.

FIGURE 5.1

ROUTING OF PMF FOR ROUND POND

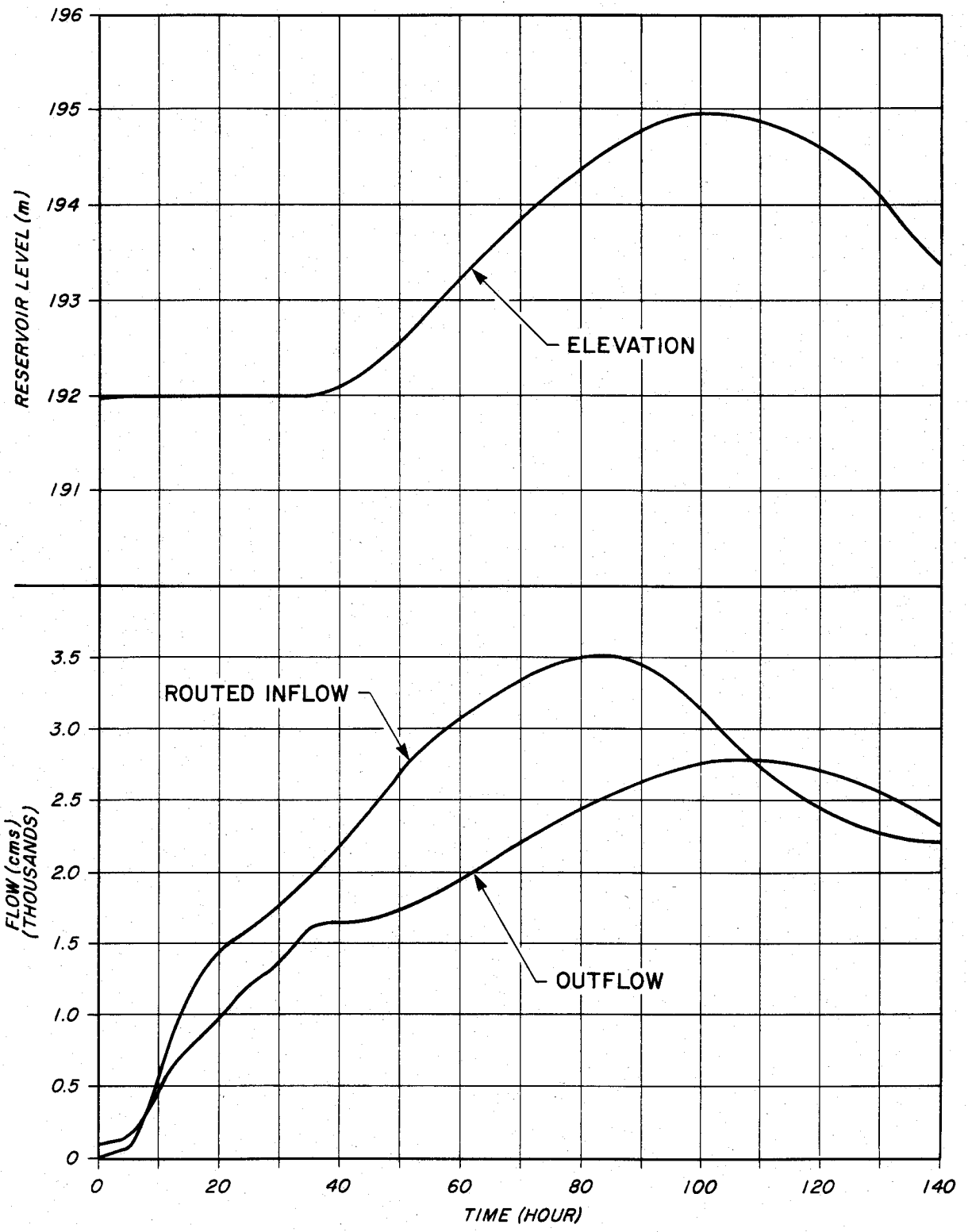
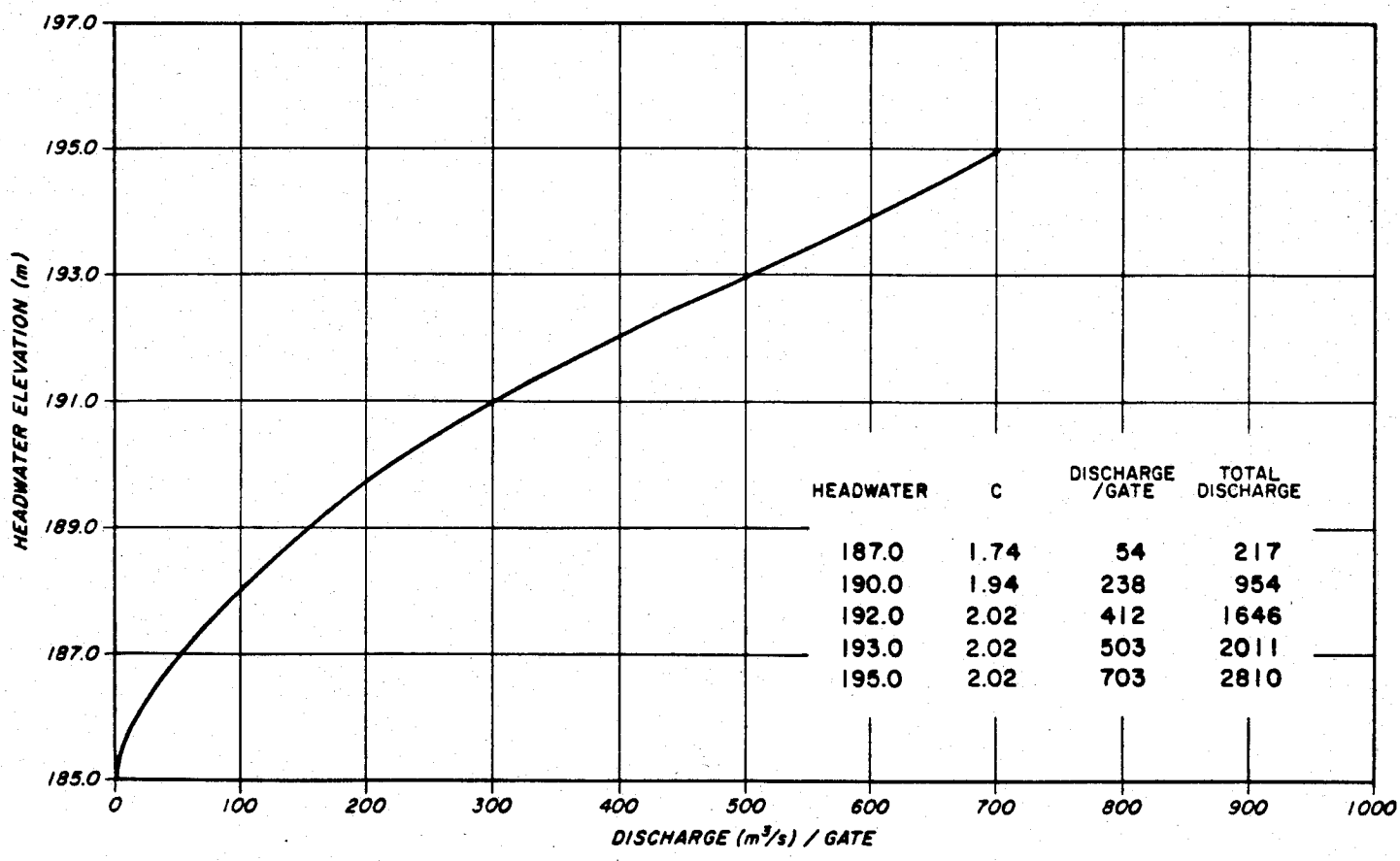


FIGURE 5.2

SPILLWAY DISCHARGE RATING CURVE



HEADWATER	C	DISCHARGE /GATE	TOTAL DISCHARGE
187.0	1.74	54	217
190.0	1.94	238	954
192.0	2.02	412	1646
193.0	2.02	503	2011
195.0	2.02	703	2810

TABLE 5.1

PMF Hydrographs for each Sub-Basin (in m^3/s)*

Time (hr)	Long Pond	Round Pond	Great Brunt Lake	Meelpaeg	Granite Lake	Burnt Pond	Victoria Lake	Cold Spring Lake	Island Pond	Spruce Pond
0	8	12	4	66	34	8	17	7	1	20
6	49	70	22	261	129	34	87	39	8	87
12	137	199	62	483	237	72	207	94	21	182
18	258	372	119	646	315	105	333	149	38	265
24	385	549	179	806	394	134	452	197	55	338
30	512	722	242	1012	495	168	588	249	74	425
36	664	927	318	1320	642	215	760	321	98	545
42	858	1193	417	1664	804	272	971	415	128	690
48	1070	1485	527	1866	898	316	1170	500	161	800
54	1254	1730	627	1949	934	338	1310	551	189	854
60	1388	1897	707	2029	970	351	1410	584	212	888
66	1491	2017	775	2117	1009	365	1503	616	232	925
72	1572	2107	835	2107	1000	369	1570	636	250	935
78	1617	2147	878	2013	949	356	1579	632	262	902
84	1620	2131	901	1933	905	339	1551	615	268	859
90	1569	2037	898	1638	766	302	1476	571	267	765
96	1437	1822	852	1179	552	236	1308	472	251	597
102	1228	1498	766	840	394	171	1100	358	225	434
108	1000	1155	668	597	280	124	917	269	197	315
114	802	873	579	424	199	90	765	202	173	228
120	644	660	502	301	141	65	638	151	151	165
126	517	499	435	213	100	47	532	113	133	120
132	415	376	377	150	70	34	443	85	116	86
138	333	283	327	104	49	24	370	63	102	62

* results from 1985 Flood Study

6. ENERGY STUDIES

6. ENERGY STUDIES

6.1 GENERAL

This section describes the background, methodology and results of the energy studies carried out for the feasibility study. The energy studies carried out during the prefeasibility study are described in Appendix B.

Optimization studies were carried out to determine the optimum Full Supply Level (FSL) and plant capacity. Optimization of the FSL became necessary, when after the prefeasibility study site investigation, it was apparent that additional available head found at the site would necessitate construction of long saddle dams, which would have a significant impact on the total capital cost of the Development.

6.2 UPPER LIMIT OF FULL SUPPLY LEVEL

The FSL of the Development was restricted to a maximum El. 192.0, to avoid encroaching on the existing water level of Godaleich Pond. An environmental study conducted before implementation of the Upper Salmon Development recommended that because of the increased flows from the power plant at Godaleich, the Lower West Salmon Outfall area at Godaleich Pond (Ref. Dwg. 2.1) would require modification to ensure passage of fish from Round Pond and also to maintain the natural water levels in Godaleich Pond to protect the delta area at the west corner of the lake as indicated on Dwg. 6.1. As a result of this study a low velocity path some 850 m long was constructed along the main channel to the left side of the outfall area.

Restriction of the FSL below or equal to El. 192.0 ensures the present water levels in Godaleich Pond are not altered. This is slightly below the average tailwater level of Upper Salmon Generating Station (El. 192.18) and 1.28 m below the maximum tailwater level (El. 193.28*).

6.3 DEFINITIONS OF FIRM AND AVERAGE ENERGY

The definitions of firm and average energy used in this study as given by HYDRO are as follows:

a) Firm Energy

The firm energy is the annual system generation based on the 34-month dry period from June 1959 to March 1962.

b) Average Energy

Average energy is defined as the average annual system generation of the 37 year period of records.

6.4 DEFINITION OF ENERGY BENEFIT

The energy benefit of the Development is defined as the difference in system generation of the Bay d'Espoir System with and without the Round Pond Development.

6.5 COST OF ALTERNATIVE POWER AND ENERGY

The value of alternative energy used in the study is $0.76/kWh in 1987 dollars and is based on the average cost per kwh of two scenarios:

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* These elevations are referred to the Long Pond Datum which is 5.483 m above the datum at Godaleich Power station.

i) The cost of Gull Island energy transmitted from Labrador to the Island via the proposed HVDC transmission.

ii) The cost of thermal replacement.

No value was given to the capacity of the Round Pond Development.

6.6 RESERVOIR OPERATION

The Round Pond plant will be operated strictly as a run-of-river development. There is no benefit in operating the plant in a peaking mode, since inflow to Round Pond is controlled by discharges from the large storage reservoirs upstream (e.g. for an average inflow of 163 m^3/s the local inflow from the Round Pond sub-catchment is 28 m^3/s) and there is minimal spill in the Bay d'Espoir system as a whole, as seen from the 37 years of records used as the data base for the study. Hence to replenish live storage in the Round Pond reservoir would deprive the Bay d'Espoir plant downstream of production capability.

It is envisaged that the reservoir will only be drawn down appreciably in an exceptionally dry period, when the upstream reservoirs are depleted. In such an event it is estimated that the minimum operating level will be 3 m below the FSL. This is considered to be the minimum water level at which the turbine can be expected to operate with an efficiency range of 80-85%.

The volume of water contained between the minimum operating level and the existing lake level (El. 186.0) has been defined as the Dead Storage Volume. The Project has been penalized with the cost of the fuel equivalent of this volume of water to compensate for lost production at the Bay d'Espoir plant during reservoir impoundment.

The energy which will be lost at Bay d'Espoir at the optimum FSL of 192.0 m as the result of the dead storage has been estimated to be 74.4 GWh. The fuel equivalent cost has been evaluated at \$2.23 x 10^6 based on 30 mills/kwh (1987 dollars), as provided by HYDRO.

The flow velocities in the intake channel are sufficiently low to allow formation of a stable ice cover, which will prevent frazil ice from developing. No provision will therefore be necessary for trashrack heating.

6.7 OPTIMIZATION STUDIES

6.7.1 Methodology

Optimization of the FSL and installed capacity was done by estimating the benefits and costs of various alternatives and selecting that alternative which resulted in the maximum benefit/cost ratio.

The costs and benefits were evaluated in accordance with the procedure described as follows:

a) Costs

The following elements were included in the cost of the proposed Development for the purposes of the optimization exercise:

- capital cost of the Development, based on the major cost elements such as the causeways, spillway, saddle dams and power facilities.

- operation and maintenance (O & M) of the Plant.

- dead storage penalty (as defined in Section 6.6).

Costs were estimated in 1987 dollars. Based on a construction schedule with an on-power date of December 1991 and a construction period of 33 months, the disbursement of capital cost was approximated to the following:

Year	Capital Cost Expressed as a Percentage
1989	9.3
1990	60.7
1991	30.0

Using an escalation of 5% a year on capital cost, the total cost of the Development on completion in 1991 was computed. The annual O & M cost was then estimated as:

(CIVIL COST) x 0.25% + (MECHANICAL/ELECTRICAL COST) x 1.35%

The miscellaneous costs such as temporary services, owners costs, etc., were distributed in proportion to the civil and mechanical/electrical costs and included in the above.

The annual O & M costs for the 60 year life of the Development were calculated with an escalation rate of 4.32% per annum. The present worth of the series was then calculated to 1987 using a discount rate of 11%.

b) Energy

Annual energy estimates were estimated by HYDRO, using the Bay d'Espoir Regulation Model for full supply levels ranging from El. 188.5 to El. 192.0. These are given in Table 6.1 together with the energy estimate of the existing system. The energy was calculated on a monthly basis from the following relationship:

G_{avg} (kWh) = 9.81 x Q x H x E x 24 x D

where: Q = average monthly turbine flow in m^3/s
H = net head in m
E = combined plant efficiency
(turb. eff. x gen. eff. x system eff.)
D = no. of days in the month.

The annual generation was taken to be the sum of the corresponding monthly values and the average of the 37 annual totals was used in the project design optimization.

The net head for the optimization exercise was approximated to 98% of the gross head and the value of the combined plant efficiency used was 85.7%. The tailwater level is governed by the normal operating level of Long Pond Reservoir which is generally constant at 180.75 m.

With reference to the results from the pre-feasibility study, nine scenarios were selected from those tabulated in Table 6.1.

6.7.2 Results of Analyses

Table 6.2 summarizes the details of each of the scenarios considered, together with the corresponding average annual energy for the development, the total estimated cost (including the escalated operation and maintenance cost), total revenue and benefit cost ratio.

From scenario Nos. RP 35, RP 46 and RP 56 which have approximately the same turbine flow, it can be seen that the most economic FSL is at El. 192. This confirms the result obtained from the prefeasibility study which indicated that it was economic to develop the full head between Godaleich Pond and Long Pond.

The marginal cost benefit ratios beyond 18 MW are small because of the small increases in generation. It was therefore concluded that an 18 MW installation with a corresponding turbine flow of 193.1 m^3/s and FSL of El. 192.0 is the optimum alternative.

6.8 ROUND POND ENERGY GENERATION

The plant energy output was based on the optimized FSL of El. 192.0, an average tailwater level of El. 180.75 and refined values for the overall plant efficiency and net head as follows:

- The value of combined plant efficiency used was 87.8% made up of the product of the following:

Turbine	- 91.4% * (weighted turbine efficiency)	
Generator	- 98%	
Transformer	- 99.5%)	
Plant Services	- 99.5%)	System Efficiency
Water Utilization	- 99%**)	

- The net head was the FSL of El. 192.0 less the tailwater less the following hydraulic losses:
  - Intake Channel
  - Trashrack
  - Intake
  - Exit Loss

A curve showing the relationship between the total of the above head losses and the turbine flow is given in Fig. 6.2.

Based on the foregoing, the average annual energy output would be as defined in Sub-section 6.7.1.

= 9.81 x Q x H x E x 24 x D
= 9.81 x 161.8 x 0.878 x 10.81 x 8760
= 132 GWh

192.00
180.75
11.25
.44
Losses 10.81

---

* Since the Round Pond plant is to be operated as run of river the weighted turbine efficiency was used in the computation of average energy. This was calculated from the flow duration curve given in Fig. 6.1.

** A water utilization factor of 99% was used as the flow exceeding 193 m^3/s in Fig. 6.1 is only 2%.

Since the proposed Round Pond Development would become an integral part of the Bay d'Espoir System, firm and secondary energy were evaluated in terms of the total system output. This was done using the Bay d'Espoir Regulation Model which utilizes a 37 year period of records. The firm energy of the Round Pond Development was calculated based on the definition given in Section 6.3.

The simulated system energy output in GWh per annum are summarized below for the Bay d'Espoir System both with and without the Round Pond Development:

System Output	Existing (without Round Pond)		
	Firm	Secondary	Total
Round Pond	--	--	--
Upper Salmon	475	65	540
Bay d'Espoir	2209	344	2553
TOTAL	2684	409	3093

System Output	Existing (With Round Pond)		
	Firm	Secondary	Total
Round Pond	109	23	132
Upper Salmon	474	66	540
Bay d'Espoir	2229	324	2553
TOTAL	2812	413	3225
TOTAL Incremental	128	4	132

The analysis shows that there would be a net benefit to the Bay d'Espoir system with the addition of the Round Pond Development.

6.9 PLANT CAPACITY

The rated plant capacity is based on the gross head less the head losses in the intake channel, intake and the tailrace at the rated flow. The rated flow for the Round Pond Development is 193 m^3/s

as determined from the optimization studies. It has been assumed that the reservoir will be maintained at El. 192.0 during normal operation. The net head is therefore given as follows:

Round Pond Reservoir Level (m)	192.0
Tailwater Level Long Pond (m)	180.75
Total Head Loss (m)	0.44
Net Head (m)	10.81

The turbine and generator efficiencies and the corresponding outputs under rated conditions are tabulated below:

Turbine Efficiency	-	93%
Generator Efficiency	-	98%
Turbine Output (MW)	-	19.1
Generator Output (MVA)	-	20.8
Installed Capacity	-	18.0

The plant capacity factor is calculated as follows:

$$\frac{\text{Average annual energy}}{\text{Maximum annual plant output}} = \frac{132 \times 10^6 \times 100}{18.0 \times 10^3 \times 8760} = 84\%$$

6.10 FLOOD CONTROL

As discussed previousy in Section 5 the structures have been designed to perform safely should the probable maximum flood (PMF) occur. During this flood a maximum level of 195.0 would be attained.

During a flood event approaching this magnitude, it is assumed that the power plant would be inoperable and the flood would be passed completely through the gated spillway.

TABLE 6.1

PROPOSED ROUND POND DEVELOPMENT AND EXISTING SYSTEM REGULATION COMPUTER RUNS

RUN	SUM & DAT FILES	R. POND FSL (m)	R. POND TWR (m)	R. POND LSL	R. POND UNIT (MW)	R. POND H.L.	R. POND HEAD	R. POND EFFEC.	ENERGY (GWH) BAY D'ESPOIR	ENERGY (GWH) UPPER SALMON	ENERGY (GWH) R. POND	ENERGY (GWH) TOTAL
ARSP.EXT	EXSTO	NA	NA	NA	NA	NA	NA	NA	2553	540	NA	3093
ARSP.R11	RP110	188.50	188.50	187.50	8	0.16	7.75	.857	2508	507	70	3085
ARSP.R12	RP120	188.50	188.50	187.50	10	0.16	7.75	.857	2538	526	84	3148
ARSP.R13	RP130	188.50	188.50	187.50	12	0.16	7.75	.857	2553	539	90	3182
ARSP.R14	RP140	188.50	188.50	187.50	14	0.16	7.75	.857	2553	540	96	3184
ARSP.R15	RP150	188.50	188.50	187.50	16	0.16	7.75	.857	2553	540	91	3184
ARSP.R21	RP210	189.50	189.50	188.50	8	0.18	8.75	.057	2496	498	70	3064
ARSP.R22	RP220	189.50	189.50	188.50	10	0.18	8.75	.057	2519	515	87	3121
ARSP.R23	RP230	189.50	189.50	188.50	12	0.18	8.75	.857	2548	532	98	3178
ARSP.R24	RP240	189.50	189.50	188.50	14	0.18	8.75	.857	2553	540	102	3195
ARSP.R25	RP250	189.50	189.50	188.50	16	0.18	8.75	.857	2553	540	103	3196
ARSP.R32	RP320	190.50	190.50	189.50	10	0.20	9.75	.857	2508	506	87	3101
ARSP.R33	RP330	190.50	190.50	189.50	12	0.20	9.75	.857	2530	521	103	3154
ARSP.R34	RP340	190.50	190.50	189.50	14	0.20	9.75	.857	2551	535	111	3197
ARSP.R35	RP350	190.50	190.50	189.50	16	0.20	9.75	.857	2553	540	114	3207
ARSP.R36	RP360	190.50	190.50	189.50	18	0.20	9.75	.857	2553	540	115	3208
ARSP.R43	RP430	191.50	191.50	190.50	12	0.22	10.75	.857	2516	513	104	3133
ARSP.R44	RP440	191.50	191.50	190.50	14	0.22	10.75	.857	2540	527	117	3184
ARSP.R45	RP450	191.50	191.50	190.50	16	0.22	10.75	.857	2553	537	124	3214
ARSP.R46	RP460	191.50	191.50	190.50	18	0.22	10.75	.857	2553	540	126	3219
ARSP.R47	RP470	191.50	191.50	190.50	20	0.22	10.75	.857	2553	540	126	3219
ARSP.R54	RP540	192.00	192.00	191.00	14	0.23	11.25	.857	2532	522	119	3173
ARSP.R55	RP550	192.00	192.00	191.00	16	0.23	11.25	.857	2550	534	128	3212
ARSP.R56	RP560	192.00	192.00	191.00	18	0.23	11.25	.857	2553	540	132	3225
ARSP.R57	RP570	192.00	192.00	191.00	20	0.23	11.25	.857	2553	540	132	3225
ARSP.R58	RP580	192.00	192.00	191.00	22	0.23	11.25	.857	2553	540	132	3225

NOTE: Results provided by HYDRO
Energy given to the nearest GWH.

TABLE 6.2

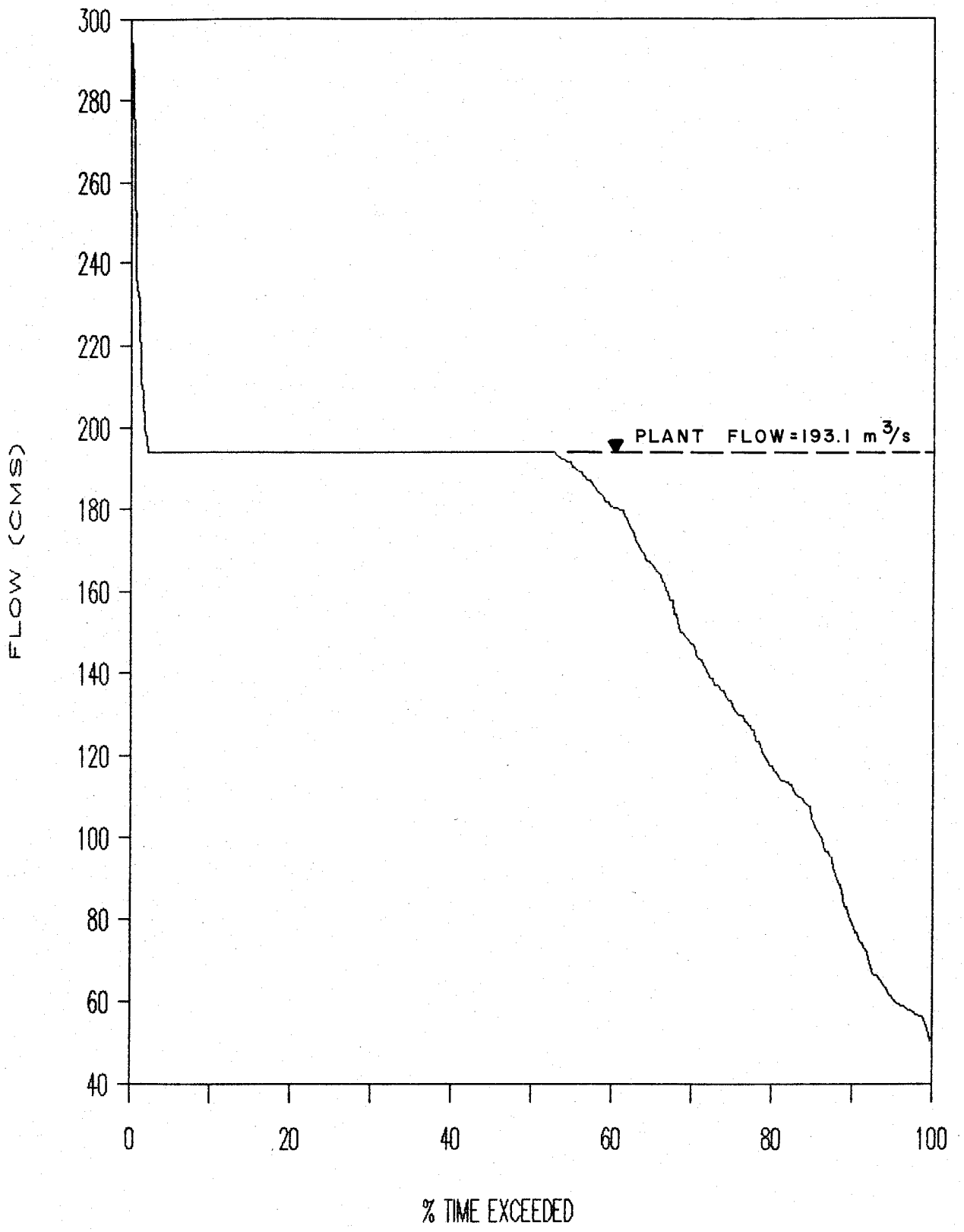
SUMMARY OF OPTIMIZATION

ID NO.	FSL (m)	MW	FLOW (cms)	ANN. RP BEN. (kWh x 10^3)	TOTAL COST ($1000)	TOT. REV. ($1000)	B/C	$/kwh
RP58	192.0	22	236.0	132200	64589	100472	1.56	0.49
RP57	192.0	20	212.0	132000	61511	100320	1.63	0.47
RP56	192.0	18	194.0	131300	57853	99788	1.72	0.44
RP55	192.0	16	173.0	119000	56550	90440	1.60	0.48
RP54	192.0	14	151.0	80200	53574	60952	1.14	0.67
RP46	191.5	18	203.2	126100	58161	95836	1.65	0.46
RP45	191.5	16	180.6	120600	55268	91656	1.66	0.46
RP36	190.5	18	224.0	114400	57005	86944	1.53	0.50
RP35	190.5	16	199.4	114100	53578	86716	1.62	0.47

FIGURE 6.1

FLOW DURATION CURVE AT ROUND POND

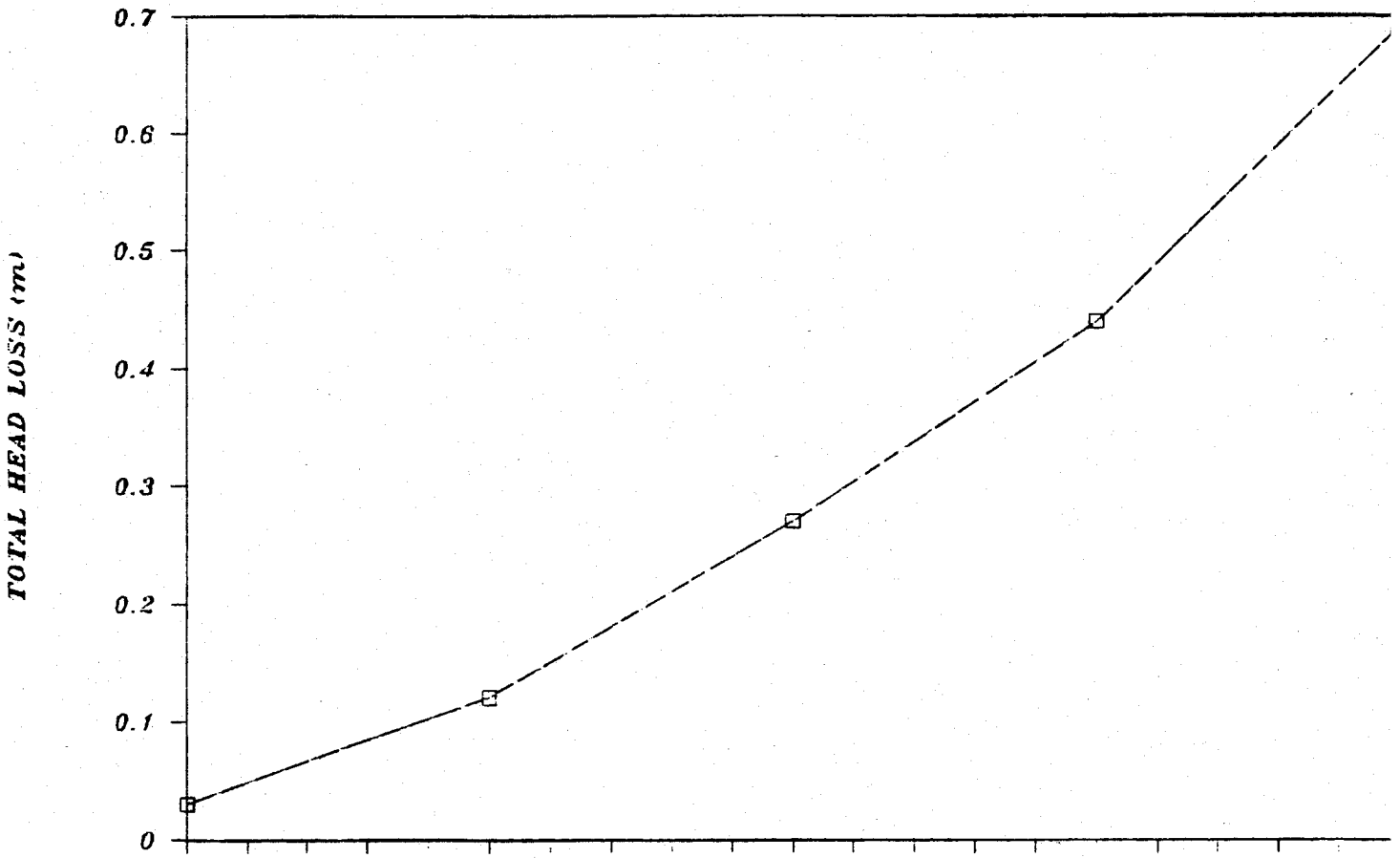
FSL 192 m 18 MW



NOTE: CURVE IS FOR POST DEVELOPMENT CONDITION

Figure 6.2

HEAD LOSS CURVE FOR OPTIMUM SCHEME



7. DESCRIPTION OF DEVELOPMENT

7. DESCRIPTION OF DEVELOPMENT

7.1 GENERAL

The layout selected is illustrated on Drawings 7.1 and 7.2 and is summarized in Section 2.4. A detailed description of each of the component structures follows in Section 7.3.

The full supply level (FSL) and installed capacity of the Development are El. 192.0 m and 18 MW respectively, as determined by the optimization procedure described in Section 6.0.

7.2 ALTERNATIVES CONSIDERED

The selected layout was chosen from several alternatives considered for the alignments of the causeways, the saddle dams and the types of generating equipment suitable for the available head at the site. Full details are provided in Appendix A at the end of this report.

7.3 SELECTED LAYOUT

7.3.1 Access Road

The access road is approximately 12 km long and follows a southwesterly direction to join the Upper Salmon Road some 19 km from the Upper Salmon Generating Station. The route is close to esker deposits located some 5 to 7 km from the Round Pond site, which are suitable for road fill materials.

One major bridge with an approximate span of 25 m is required some 2 km from the Round Pond site.

The road will be gravel surfaced, 6 m wide and will be suitable for transportation of heavy equipment to the site.

7.3.2 Dams

7.3.2.1 Causeways

The causeways are approximately 800 m long and extend from the access road on the right side of the west channel to the powerhouse on the extreme south side of the project area.

The dam is a homogeneous construction and comprises compacted glacial till with a rip rap face extending from El. 188 to crest level to protect the upstream side. A transition zone separates the rockfill from the glacial till. A filter layer is provided on the downstream side at foundation level to lower the phreatic line near the base of the dam to prevent saturation of the downstream shell.

Slopes of 2.5 horizontally to 1 vertically have been adopted on both the upstream and downstream sides. A cut-off trench is excavated to a suitable foundation and filled with compacted till.

The choice of dam was dictated by the availability of materials at the site. Insufficient rock was available from the structure excavations to warrant having a zoned rockfill construction for the dam. Large till deposits suitable for use in a homogeneous dam, are however available close to the site.

The glacial till for the dam will be selected from borrow areas T-1 and T-3 adjacent to the site and the transition material will be processed from granular material obtained from the esker deposits 5 to 7 km south of the site.

The crest width of the causeway embankment is 6.0 m and the crest elevation is 196.5 m. An allowance has been made for a 4.5 m freeboard above FSL, which includes a 3 m surcharge to allow the design flood to be discharged through the spillway and an allowance of 1.5 m to take account of wave height, set up and run up from the

maximum flood level. The latter was calculated using the method documented by Saville McClendon and Cochran (Ref. 3) assuming a wind speed of 100 km/hr.

A profile through the causeway and a typical cross section are given on Dwg. 7.3.

7.3.2.2 Saddle Dams

The saddle dam locations are illustrated on Dwg. 7.1. Saddle dams 1 and 2 are small freeboard structures with a maximum height of approximately 2.0 m. Saddle Dam No. 3 is a larger structure with a maximum height of 7.5 m and a length of approximately 1050 m.

The construction of the saddle dam is similar to the causeway, described in the previous section.

The crest elevation of the saddle dams is 196.5 m, giving a freeboard allowance of 4.5 m above FSL, as for the causeways. The crest width has however been reduced to 5 m since little traffic is expected to pass over the saddle dams.

7.3.3 Spillway

The spillway selected for the Development is a concrete low ogee gated structure which is illustrated on Dwg. 7.4.

The crest of the structure is at El. 196.5 and the overall length is 53 m. Discharge through the spillway will be regulated by four vertical lift gates 7.5 m high by 11.0 m clear wide, which are raised and lowered from a fixed hoist structure by two cables located at either end of the gates. The gates span between 3.0 m wide reinforced concrete piers. Each gate has an individual hoisting mechanism which is housed in a weatherproof structure spanning between structural steel towers. Access to the hoists is from ladders housed inside weatherproof chambers at the end towers. A

bridge 4.5 m wide is provided on the downstream side of the piers for vehicle access across the spillway. The bridge comprises precast concrete beams with a cast-in-place concrete deck.

A monorail is provided along the length of the structure for moving the stoplogs into position from a storage area located on the left side of the structure.

The profile of the spillway crest was based on the USBR - Low Ogee Crest (Ref. 4) and the invert was selected as El. 185.0 m. The spillway capacity is 2790 m^3/s, which is the routed outflow of the PMF through Round Pond. The rating curve developed for the spillway is given in Fig. 5.2.

7.3.4 Power Facilities

7.3.4.1 Intake Channel

The intake channel is 290 m long and leads from the east channel of the Round Pond outlet immediately upstream of the causeway, to the powerhouse.

The invert of the channel is at El. 179.5 m for approximately 200 m of its length and the remainder slopes from 179.5 m to the original river bed. The channel has an invert width of 22.0 m for the length of channel where the invert elevation is 179.5 m and for the remaining length the width varies from 22.0 m to approximately 50 m where it meets the original river channel. The channel is excavated mainly in glacial till material, which on the basis of the borehole data is expected to be compacted to very dense for the full depth of the channel excavation. The maximum velocity in the channel will be around 0.5 m/s, which is low enough to prevent degradation of this type of material and therefore no lining of the canal has been provided. This velocity is also sufficiently low to allow formation of a stable ice cover which will prevent frazil ice from developing.

Side slopes in the rock cut are 5V:1H and in overburden excavation are 2H:1V.

A profile and cross sections through the canal are shown on Dwg. 7.7.

7.3.5 Powerhouse

The powerhouse is located to the south-east of the east channel at the Round Pond outlet. A short tailrace channel connects the powerhouse with Long Pond Reservoir.

The structure is approximately 48 m long and 12.5 m wide and contains a single Pit 18 MW turbine. It is mainly of reinforced concrete construction and is contained within a deep cut in rock and overburden. Concrete gravity walls approximately 15 m high are provided on either side of the powerhouse to retain the dam fill.

The downstream wall of the structure is constructed of a structural steel frame and a flat built-up roof is supported on steel trusses spanning between concrete side walls.

The main access to the powerhouse is located on the right side of the downstream end at El. 186.0 m. A ramp leads from this elevation to the causeway embankment crest at 196.5 m.

The control room is located below the main access at El. 178.3 m and will house the equipment described in Sub-Section 7.3.7.

The governor pumping unit, lubricating system and closing weight/servomotor will be located in a chamber below the runner housing.

The following major mechanical equipment is provided in the powerhouse:

- a 40 tonne gantry crane for installation and maintenance of the major mechanical and electrical equipment.

- a draft tube gate approximately 10.0 m wide by 7.0 m high.

- a set of intake stoplogs for an approximate opening size of 10.9 m wide by 10.0 m high.

The draft tube gate will be raised and lowered using the powerhouse crane and will be stored in a dogged position in a slot in the powerhouse concrete sub-structure above the draft tube opening.

The stoplogs will be placed in position by a mobile crane or alternatively by a fixed monorail hoist located at the deck level at El. 196.5. The layout of the powerhouse and tailrace are shown on Dwg. 7.5. The powerhouse details are shown on Dwg. 7.6.

7.3.6. Turbine and Generators

The turbine is a single Pit unit with the following characteristics:

Net Head	- 10.8 m
Design Flow	- 193.1 m^3/s
Rotational Speed of Turbine	- 129 rpm
Rotational Speed of Generator	- 600 rpm
Runner Dia.	- 4.75 m
Turbine Output	- 19.10 MW
Approximate Efficiency	- 93%
Approximate Setting to Centre line of propellor (measured below tailwater)	- 6.0 m

The generator is a synchronous unit with the following characteristics:

- voltage 13.8 kV
- speed 600 rpm
- p.f. 0.9
- Generator output 20.8 MVA
- efficiency 98%

7.3.7 Powerhouse Electrical System

The electrical system for the powerhouse is illustrated on Dwg. 7.8.

The generator output will be connected to the main step-up transformer through a 13.8 kV vacuum or SF6 withdrawable type circuit breaker located in the control room near the automatic control board used for the remote operation of the plant.

The DC system will be supplied from a calcium (sealed type) lead acid battery.

Station services will be provided through a distribution board by a dry type transformer, fed from 13.8 kV bus through a metal enclosed disconnect switch.

The following equipment will be provided and located in the control room.

1) 13.8 kV switchgear containing the following:

- generator breaker panel
- generator excitation (transformer and rectifier, automatic voltage regulator, field breaker, etc., in two panels).
- Main feeder panel containing line side voltage transformers and surge divertors.
- Station service transformer disconnect switch
- Station service transformer

- Generator side voltage transformers and grounding switch.

2) Control and protection panels containing the following:

- line and transformer protection panel
- generator protection panel
- control panels (remote/local)

3) Station service board.

4) Station D.C. battery housed in a separate enclosure.

5) Battery charger.

6) D.C. distribution board.

The governor control panel, turbine/generator panel and generator neutral grounding cubicle will be located on the main floor at El. 178.3.

A standby diesel generator will be provided in the repair bay area adjacent to the downstream wall to facilitate venting.

7.3.8 Switchyard

The switchyard is located at the southwest corner of the powerhouse building and adjacent to the main access. To minimize the quantity of excavation and fill the elevation of the yard has been set at El. 186.0.

The switchyard contains the following equipment:

- ONAN/ONAF transformer
- 69 kV disconnect/ground switch
- instrument transformers
- take-off structure

The main step-up transformer, line control and protection panel will be located in the control room in the powerhouse.

The switchyard layout is shown on Drawing 7.5.

7.3.9 Fish Passage Facility

The location and type of fish passage facility will form part of a separate environmental study to be carried out on the Development. However, for the purpose of the feasibility study it has been assumed that a simple stepped fish passage will be provided alongside the powerhouse on the left side.

The fish passage facility comprises stilling pools 2.2 m long and 1.5 m wide with 0.5 m steps. The intake to the fish passage is adjacent to the power intake and the discharge is downstream of the draft tube outlet. In order to prevent fish from entering the intake and draft tube, fish screens will be provided, if necessary across these areas. Velocities in the intake area and tailrace are in the order of 0.5 m/s and 2.4 m/s respectively.

Details of the fish passage facility are given on Dwg. 7.5.

8. CONSTRUCTION

8. CONSTRUCTION

8.1 CONSTRUCTION PLANNING.

The overall master schedule for engineering and construction is shown in Fig. 8.1. The engineering program is based on the assumption of competitive tendering. Accordingly in order to ensure that the best prices are gained, by giving scope to the specialist activities of manufacturers and by providing opportunities to local firms, it is proposed to divide the work into the following contract packages:

Contract No. 1	Reservoir Clearing
Contract No. 2	Permanent Access Road
Contract No. 3	Main Civil Works
Contract No. 4	Hydraulic Gates
Contract No. 5	Generating Plant
Contract No. 6	Miscellaneous Electrical and Mechanical Equipment
Contract No. 7	Telecontrol Equipment
Contract No. 8	Transmission Lines System.

All major equipment contracts will cover manufacture, delivery, erection, testing and commissioning and setting to work of plant and equipment offered.

Further elucidation of the contract packages are provided in the following Sub-Section 8.2 CONTRACT PACKAGES.

The following durations have been included in the master schedule for each stage of the tender process for each of the above contracts:

Description	Duration
Contract No. 1 - Reservoir Clearing	
- Prequalification	1 month
- Tendering	1 month
- Evaluation & Award	1½ month
Contract No. 2 - Permanent Access Road	
- Prequalification	1 month
- Tendering	1 month
- Evaluation & Award	1 month
Contract No. 3 - Main Civil Works	
- Prequalification	2 months
- Tendering	1½ months
- Evaluation & Award	1½ months
Contract No. 4 - Hydraulic Gates	
- Prequalification	1 month
- Tendering	1½ months
- Evaluation & Award	1 month
Contract No. 5 - Generating Plant	
- Prequalification	2 months
- Tendering	3½ months
- Evaluation & Award	2½ months
Contract No. 6 - Miscellaneous Electrical and Mechanical Equipment	
- Prequalification	1 month
- Tendering	2 months
- Evaluation & Award	2 months

Description	Duration
Contract No. 7 - Telecontrol Equipment	
- Prequalification	1 month
- Tendering	1½ months
- Evaluation & Award	2 months
Contract No. 8 - Transmission Line System	
- Prequalification	1 month
- Tendering	1½ months
- Evaluation & Award	2 months

8.2 CONTRACT PACKAGES

The works envisaged under the Development have been suitably divided into the various contract packages as described in Sub-Section 8.1.

The proximity of the structural elements of the proposed Development to each other and the absence of abnormal foundation conditions which may require specialized treatment serve to retain the civil works within a single contract package. However, because the permanent access road leading to the site of the Development is required at an early stage of the implementation program and reservoir clearing requires specialized knowledge, plant and equipment, these two items of work are identified in separate contract packages apart from other civil works.

In consideration of the small hydro plant for the proposed Development it is proposed that the electro/mechanical works for the generating unit be put under a single contract package, i.e. a "water to wire" contract approach. This approach will therefore require discussion with HYDRO.

Subject to consultation with HYDRO, it is proposed that the Development should contain eight contract packages shown in the following table. This also provides a brief description of the scope of work within each package.

CONTRACT PACKAGES

CONTRACT NO.	CONTRACT TITLE	SCOPE OF WORK
1	Reservoir Clearing	- Land clearing to El. 193.0 and salvage/ disposal of timber.
2	Permanent Access Road	- Construction of road from the existing Upper Salmon Road to the Work Site. - Construction of one bridge and pipe culverts for stream crossings.
3	Main Civil Works	- Provision of construction camp, temporary facilities and utilities. - Construction of main dam and saddle dams. - Construction of concrete spillway dam - Installation embedded parts, under manufacturer' instruction and supervision, for hydraulic gates and stoplogs. - Construction of power intake canal, power-house, tailrace and switchyard. - Installation of powerhouse crane under manufacturer's instruction and supervision. - Construction of fish pass.
4	Hydraulic Gates	Design, manufacture and deliver to Site, - spillway gates, spillway stoplogs, including embedded parts and hoist equipment - support and housing to spillway gates. - powerhouse draft tube gates including embedded parts and hoist equipment. - powerhouse upstream stoplogs and trashracks. Erection, except embedded parts, and testing of plant.

CONTRACT NO.	CONTRACT TITLE	SCOPE OF WORK
5	Generating Plant	Design, manufacture, deliver to Site, install and commission: - Pit turbine - synchronous generator - main transformer - switchgear - ancillary electrical and mechanical equipment Design, manufacture, deliver to Site and Supervision of erection and testing. - powerhouse crane
6	Miscellaneous Electrical and Mechanical Equipment	Design, manufacture, deliver to Site, install and commission: - service water system. - cooling water system. - potable water system. - firefighting system. - emergency generator. - heating and ventilation. - lighting.
7 (1)	Telecontrol Equipment	
8 (1)	Transmission Lines System	

(1) By Owner.

Contract Nos. 1, 2 and 3 are expected to include a large component of local participation as a matter of course. Contract Nos. 4 and 5, because of the specialized nature of the work to be executed, may preclude local participation. In consultation with HYDRO the extent of local content in all contracts may be defined during prequalification of tenderers and formalized in the contract documents.

Provisions for telecontrol equipment (Contract No. 7) and transmission lines system (Contract No. 8), are not within the scope of the feasibility study but are nevertheless identified in the Contract Packages listed in the above tabulation for completeness.

8.3 IMPLEMENTATION SCHEDULE

In order to achieve the on-power date of December 15, YEAR 3, it will be necessary to implement award of contracts and construction of the works to the program shown in Fig. 8.1 - Engineering and Construction Master Schedule. The Master Schedule is designed to maximize the benefit of carrying out engineering, procurement and construction in an orderly manner. Accordingly, key dates for implementation of the Development are as follows:

HYDRO authorization to proceed with preliminary engineering	April 1,	Year 1
Project release date	July 1,?	Year 1
Approval and issue of tenders for Permanent Access Road Contract	June 1,	Year 1
Approval and issue of tenders for Main Civil Works Contract	July 1,	Year 1

Award of Main Civil Works Contract	October 1,	Year 1
Approval and issue of tenders for Hydraulic Gates Contract	September 15,	Year 1
Award of Hydraulic Gates Contract	December 1,	Year 1
Approval and issue of tenders for Generating Plant	August 1,	Year 1
Award of Generating Plant Contract	February 1,	Year 2
Approval and issue of tenders for Miscellaneous Electrical & Mechanical Equipment Contract	June 1,	Year 2
Award of Miscellaneous Electrical & Mechanical Equipment Contract	October 1,	Year 2
Start of Reservoir Impounding	July 1,	Year 3
Works Ready to Admit Water	October 1,	Year 3
On-power Date	December 15,	Year 3

8.4 CONSTRUCTION SCHEDULE

The overall program for the construction of Round Pond Development recognizes a total duration of 33 months for engineering and construction work. This time span includes 10 months for engineering, tendering and award and overlapping with 28 months for site works and construction management. The period for site works includes two winter seasons during which construction activities can be expected to be curtailed.

The Engineering and Construction Master Schedule features the on-power date as December 15, Year 3 and indicates this date could be achieved providing an initial commitment to proceed with the Development is made by April 1, Year 1 or at an earlier date.

Engineering design and preparation of tender documents should start on March 1, Year 1 for the Permanent Access Road in order to take advantage of the full Year 1 construction season. Preliminary engineering design and preparation of tender documents will start immediately upon authorization to proceed on April 1, Year 1 for the main Civil Works and Generating Plant Contracts followed three months later for similar activities relating to the Hydraulic Gates Contract.

In order to provide access to the site in advance of the Main Civil Works Contract, it is planned that the Permanent Access Road Contract be awarded by September 1, Year 1. The route of this access road has previously been identified by HYDRO. It essentially runs in a north-easterly direction from a point on the existing Upper Salmon Road about 19 Km from the Godaleich Generating Station. The entire length, some 12 km, of this permanent access should be completed by the end of YEAR 1.

The Main Civil Works Contract will be required to provide a construction camp and all temporary service utilities for the Contractor's own work force and those of other contracts for the Development. This is described in Sub-Section 8.8: TEMPORARY CONSTRUCTION FACILITIES. Service utilities provided will also include temporary electric power for installation and commissioning of plant and equipment for the Development.

The completion key dates to be achieved for the Main Civil Works Contract are:

(1)	Completion of dam structures ready to start reservoir impounding	July 1,	Year 3
(2)	Completion of power structures ready to admit water	October 1,	Year 3

In order to attain these dates the Main Civil Works Contract should be awarded by October 1, Year 1. This will provide 24 months for construction including two winter seasons. The construction planning for the Main Civil Works therefore centres round these two key dates.

To attain the key date for impounding, the plan provides for the construction of related structures in the following sequence as illustrated on Fig. 8.1.

First Construction Season:

(1) Diversion through the west channel and construct Spillway behind a cofferdam in the east channel.

(2) Construction of East Causeway, Centre Causeway and Saddle dams. It is to be noted that the East Causeway and Centre Causeway can only tie into the Spillway Dam on the completion of the abutments and wingwalls.

(3) Install and test spillway gates and stoplogs.

(4) Remove spillway cofferdam and divert water through completed Spillway. Install west channel cofferdam to El. 188.0.

Second Construction Season:

(5) Construct West Causeway to EL. 191.0 by July 1, Year 3.

(6) Commence impounding on July 1, Year 3 to El. 191.0.

(7) Complete West Causeway to full height El. 196.50 m by October 1, Year 3.

(8) Complete impounding to Full Supply Level at El. 192.0 by October 1, Year 3.

It is essential that reservoir impoundment be carried out in a controlled manner to minimize the loss in production at the Bay d'Espoir plant downstream as well as for reasons of safety during construction. This factor dictates an early start and completion of the Spillway Dam with all gates and stoplogs fully tested.

The construction and completion of power structures, namely the power canal, intake/powerhouse and tailrace to meet the key date to admit water on October 1, Year 3 is straightforward with site work executed during the two construction seasons and planned to interface with installation of the generating unit and other equipment. It is planned that the power canal will be excavated during the first construction season behind a natural plug at the head of the canal. This plug will be removed after completion of the canal immediately before the key date or at an earlier time if necessary. The construction of the powerhouse is straightforward and will take place during the two-year construction period. Although the powerhouse reaches about 20 m below original ground level the ground conditions are expected to be good and open excavation should present no major problems. Concrete is placed in two stages to interface with installation of plant and equipment - first stage concrete to set in position equipment embedded parts and second stage concrete to set in position equipment installed after first stage concrete. Erection of plant and equipment takes place with the power canal in a dry state. To provide a degree of flexibility this work may also be carried out behind stoplogs installed in the intake and the power canal filled with water.

A tentative time schedule for land clearing of the reservoir area to the limits described in Sub-Section, 9.2 has been included.

8.5 EQUIPMENT SCHEDULE

Plant and equipment to be provided for the Development is encompassed within two contract packages - for generating plant and for hydraulic gates. It is expected that the provision of power generation equipment from a single source would produce efficiencies of scale and minimize coordination delays to meet the site delivery dates of major components. The delivery periods as scheduled in the Engineering and Construction Master Schedule for major components of each contract package are tabulated below:

Description	Delivery Period from Receipt of Order
Generating Plant Contract	
- Pit Turbine and Generator	16 months
- Transformer and Switchgear	18 months
- Ancillary Electrical & Mechanical Equipment	18 months
- Powerhouse Crane	14 months
Hydraulic Gates Contract	
- Spillway Gates, Stoplogs and Associated Equipment	9 months
- Draft Tube Gates and Associated Equipment	18 months
Miscellaneous Electrical and Mechanical Equipment Contract	
- Firefighting Systems, Water Systems, Emergency Generator and other Equipment	9 months

In general no major problems in meeting equipment delivery dates are forecast particularly if these orders are covered at all stages by adequate inspection and expediting controls.

8.6 MATERIALS

The project Civil Works consists primarily of excavation and fill and of concrete works. The approximate quantities of such work are summarized as follows:

Description	Unit	Quantity
Earth excavation	m^3	172,000
Rock Excavation	m^3	28,000
Impervious Fill	m^3	366,500
Filters	m^3	77,800
Rockfill	m^3	92,800
Concrete	m^3	21,900
Reinforcing Steel Bar	tonne	1,300

Impervious fill will be obtained from excavation or from till deposits described in Section 4 of this report.

Rock obtained from excavation and rock and sand obtained from borrow areas will provide the sources, after due processing, for coarse and fine filter materials, for other types of rockfill, and aggregates for concrete. These borrow areas are described in Section 4 of this report. Concrete will be site-mixed.

8.7 MANPOWER REQUIREMENTS

An estimate of the manpower requirements has been prepared based on the Engineering and Construction Master Schedule. It assumes that work will be carried out on single 10 hour shifts per day for 6 working days a week excluding legal holidays.

The following tabulation summarizes the total peak requirements for each year of the construction period. These figures include all

construction work except telecontrol equipment and transmission lines. Estimated manpower requirements on a monthly basis during construction are shown in Fig. 8.2.

Construction Year	Peak Manpower Requirement
1989	235 manmonths
1990	1235 manmonths
1991	645 manmonths
TOTAL	2115 manmonths

It can be expected that the labour content will be filled in most parts by the general construction trades to be found locally.

8.8 TEMPORARY CONSTRUCTION FACILITIES

8.8.1 Construction Camp

It is envisaged that accommodation at the camp site will be restricted to personnel on single status. This will comprise bunkhouse type trailer units for the main work force and smaller units with separate sleeping and living accommodation and communal kitchen/dining facilities for engineering staff, Owner's representative and equipment erectors.

Canteen facilities will be provided for the main work force. The camp area will also include first aid, warehouse facilities and site offices.

A suitable site for the camp appears to lie on the north-east bank of Ahwachanjeesh Pond, some 5 km south west of the site. A water intake and treatment plant for the camp water supply would be

located at the shore of the lake. A sewage treatment plant would be located so as to discharge the sewage effluent from the camp, downstream of the potable water intake.

Power for the construction camp and for construction power would be supplied from diesel generator sets installed in the camp area.

8.8.2 Temporary Construction Roads

Because of the sequence of construction adopted for the study, temporary construction roads and bridges will be necessary to gain access to the west side of the project area. This temporary access will be located to the south of the site to keep construction traffic clear of the construction of the Causeway and the Power Intake Channel.

The temporary access will comprise the following:

- a Bailey bridge with two 25 m spans over the west channel at the base of the rapids. A timber crib support protected by a rock groin will be required in the centre of the channel.

- a single span Bailey bridge with a 25 m span over the east channel,

- approximately 800 m of temporary access roads connecting the permanent access road with the powerhouse site.

Short spurs from the construction road would be required to reach the quarry area and borrow areas T-1 and T-3.

8.8.3 Temporary Channel Diversion

The scheme proposed for diversion during construction is shown on Dwg. 8.1. Of all the alternatives investigated this was considered to offer the best security for the works during construction.

It is proposed to commence construction with the spillway in the east channel. Flow will be diverted through the west channel by constructing Cofferdam No. 1 in the east channel to El. 188 over the northern tip of the island. This will protect the causeway foundation, the rock quarry and the temporary construction bridge on the east side of the island in addition to the spillway site itself against the 1 in 20 year flood.

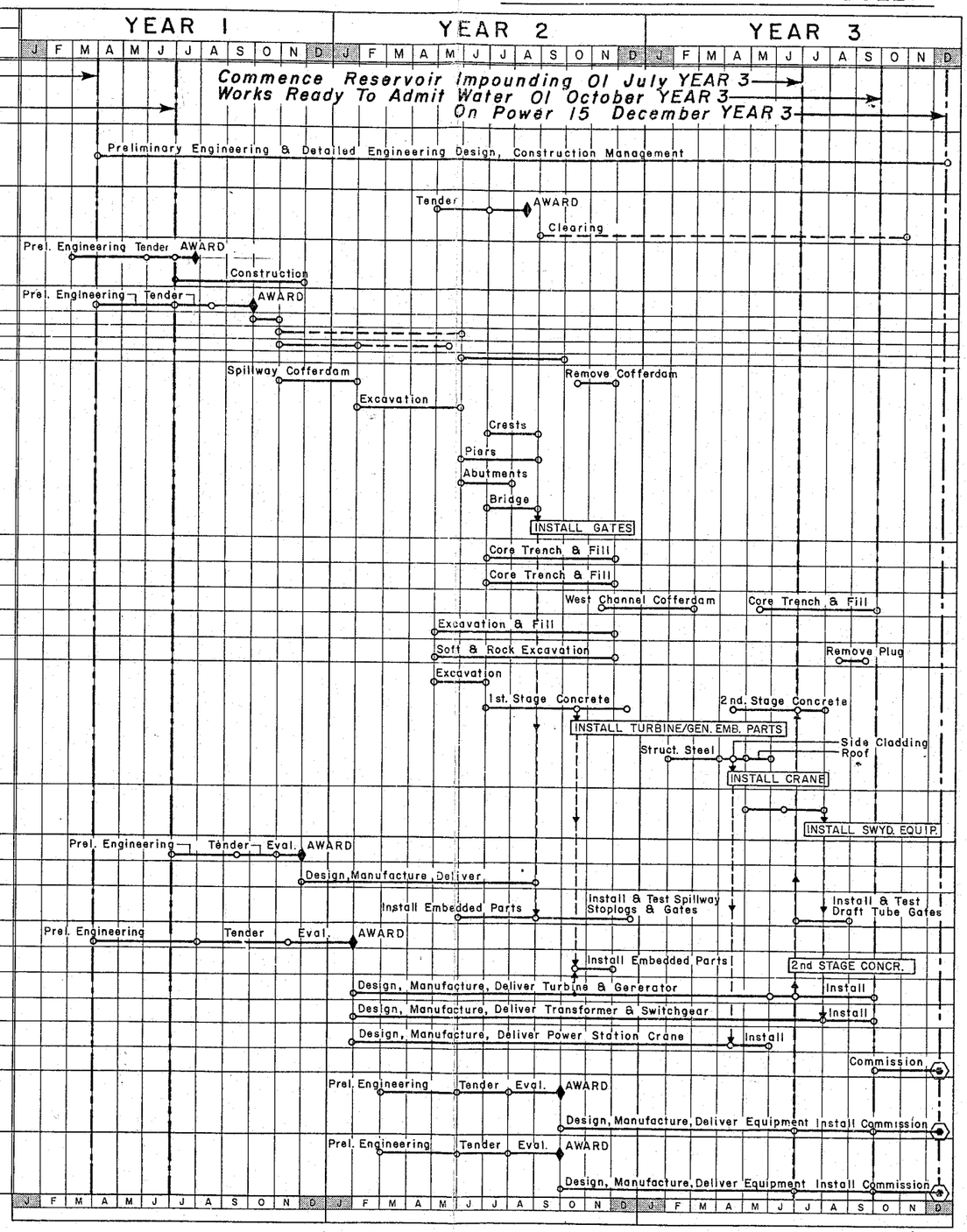
On completion of the spillway, Cofferdam No. 1 will be removed and the west channel blocked off by constructing Cofferdam No. 2, allowing the total flow to pass through the completed spillway. A plug will be left at the upstream end of the power intake channel to allow construction to continue in the powerhouse in dry conditions. This will be removed prior to impoundment.

FIGURE 8.1

ROUND POND HYDROELECTRIC DEVELOPMENT FEASIBILITY STUDY

- MASTER ENGINEERING & CONSTRUCTION SCHEDULE -

CONTRACT No.	CONTRACT TITLE	WORK ITEMS	MAJOR QUANTITIES CLEARING ha	SOFT EXCAV. m^3	ROCK EXCAV. m^3	IMPERV. FILL m^3	FILTERS m^3	ROCKFILL m^3	CONCRETE m^3
	PROJECT KEY DATES	*Approval to proceed with preliminary engineering 01 April YEAR 1*							
		Project release date 01 July YEAR 1							
	ENGINEERING & CONSTRUCTION MANAGEMENT	-PREL. & DETAILED ENGINEERING DESIGN -TENDER DOCUMENTATION -TENDER EVALUATION -ENGINEERING SUPPORT AFTER CONTRACT AWARD -CONSTRUCTION MANAGEMENT							
1	RESERVOIR CLEARING		470	-	-	-	-	-	-
2	ACCESS ROADS		12 km of roadworks						
3	CIVIL WORKS	PREL. ENGINEERING & TENDERING							
		MOBILISATION							
		CONSTR. CAMP & TEMPORARY FACILITIES							
		TEMPORARY BRIDGES & SITE ACCESS ROADS							
		FISH PASS	-	-	-	-	-	-	2200
		SPILLWAY	-	3200	2100	-	-	-	8500
		EAST CAUSEWAY	8	8400	5000	74000	7000	10500	-
		CENTRE CAUSEWAY	8	12600	5000	96000	10500	11200	-
		WEST CAUSEWAY	4	10800	4500	87000	8800	9500	-
		SADDLE DAMS	3	45000	15000	102000	23500	25000	-
		POWER CANAL	5	155000	12000	-	-	-	-
		POWERHOUSE	-	49000	8000	-	-	-	10800
		SWITCHYARD	-	3500	-	-	-	2100	400
4	HYDRAULIC GATES	PREL. ENGINEERING & TENDERING							
		SPILLWAY GATES & STOPLOGS	4 SETS SPILLWAY GATES & 1 SET STOPLOGS						
		POWER STATION DRAFT TUBE GATES	1 SET DRAFT TUBE GATES & 1 SET INTAKE STOPLOGS						
5	GENERATING PLANT	PREL. ENGINEERING & TENDERING							
		TURBINE							
		TURBINE / GENERATOR	1 UNIT PIT TURBINE / GENERATOR						
		TRANSFORMER / SWITCHGEAR	1 UNIT POWER TRANSFORMER						
		ANCILLIARY EQUIP./POWER STAT. CRANE	1 UNIT POWER STATION CRANE						
6	MISC. ELECTRICAL AND MECHANICAL EQUIP.	WATER SYSTEMS FIREFIGHTING SYSTEM EMERGENCY GENERATOR HEATING AND VENTILATION LIGHTING							
7	TELECONTROL EQUIPMENT								
8	TRANSMISSION LINES SYSTEM-UNDER OWNER SUPPLY AND CONTRACT ADMINISTRATION								

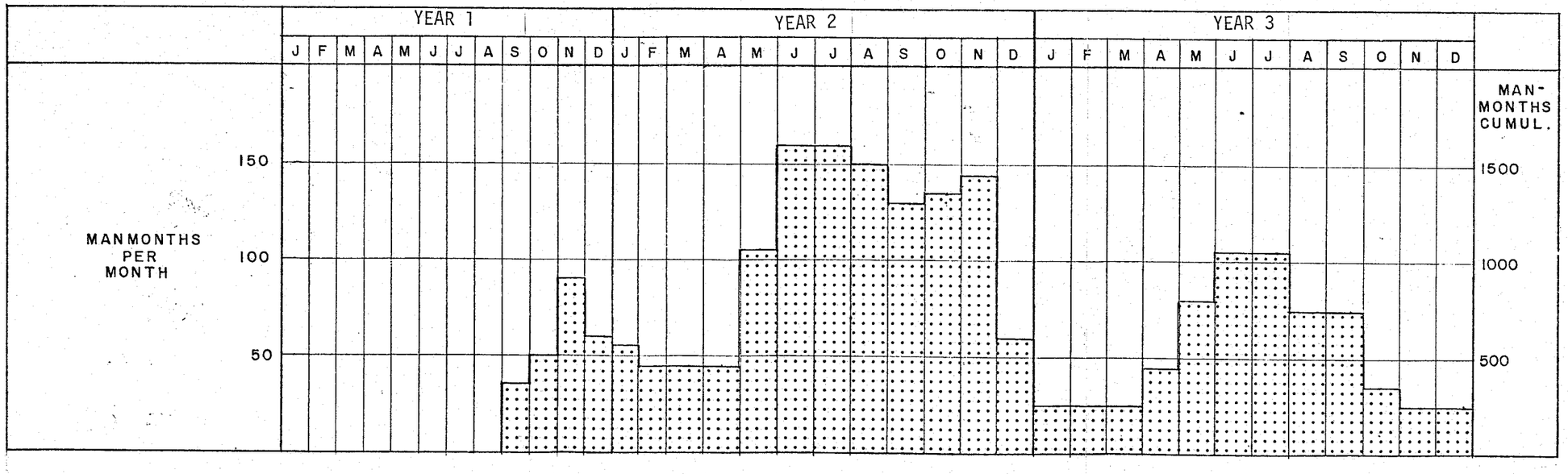


ISSUED: JUNE 88, SEPT. 88 (1)

FIGURE 8.2

ESTIMATED MANPOWER REQUIREMENT CHART

CONTRACT NO.	CONTRACT TITLE	YEAR 1												YEAR 2												YEAR 3												TOTAL MAN-MONTHS
		J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	
1	Reservoir Clearing																					10	10	10						10	10	10	10	10	10			90
2	Access Road									35	35	35																										105
3	Civil Works										15	55	55	60	66	45	45	45	105	155	155	145	115	115	125	55	25	25	25	40	65	65	60	10	10	5	5	1630
4	Hydraulic Gates																		5	5	5	5	5	5	5	5								5	5			45
5	Generating Plant																						5	5					5	5	25	25	40	40	15	15	15	195
6	Miscellaneous Electrical and Mechanical Equipment																														5	10	10	10	5	5	5	50
7	Remote Supervisory Equipment					(U	N	D	E	R		O	W	N	E	R'	S	R	E	S	P	O	N	S	I	B	I	L	I	T	Y)								
8	Transmission Lines System					(U	N	D	E	R		O	W	N	E	R'	S	R	E	S	P	O	N	S	I	B	I	L	I	T	Y)								
	TOTAL									35	50	90	60	55	45	45	45	105	160	160	150	130	135	145	60	25	25	25	45	80	105	105	75	75	35	25	25	2115



9. CAPITAL COST ESTIMATES

9. CAPITAL COST ESTIMATES

9.1 GENERAL

Capital cost estimates for the Development have been made with due consideration of the proposed implementation schedule and recognition of local conditions. Every effort has therefore been made to prepare estimates to an acceptable level of accuracy.

Three levels of cost estimates were prepared as follows:

1) Preparation of a prefeasibility level of project costs for the purpose of selecting the most advantageous layouts. These estimates were based on unit costs for the principal items of work which were anticipated. The level of accuracy was consistent with the data which was available.

2) Preparation of preliminary capital costs directed towards the preliminary optimization of the plant flow, capacity and benefit for each alternative investigated. These estimates were based on a preliminary design of the key project elements and were made with the topographical and geological field work still only partially completed. Consequently, the level of detail was less than that which was adopted for the preparation of the final estimates. This intermediate step was necessary to save time in obtaining the final quantities and the detailed cost estimates when all the field data became available.

3) After the receipt of all the topographical and geological data, the design was finalized and the quantities which were obtained in the preliminary design were adjusted to suit the field data. These detailed costs were then used to finalize the capital cost estimate for the Project.

The capital cost estimates, given in Table 9.1, indicate average figures which are expected to lie within the range of low and high bids. The estimates reflect cost trends in labour, materials, construction equipment, electro-mechanical equipment and trends in unit construction prices which have been observed in the Province and elsewhere as applicable. In the case of the electro-mechanical equipment, target estimates were also obtained from manufacturers and suppliers.

It has been assumed that contracts for major civil works, electrical and mechanical equipment will be awarded after the evaluation of open, competitive bidding.

9.2 BASIS FOR ESTIMATES

All costs which are presented in this report, unless specifically identified to the contrary, are expressed in July 1988 price levels.

Although, the contract for the Permanent Access Road is under HYDRO management nevertheless the cost of construction has been included in the Capital Cost Estimates based on quantities of work provided by HYDRO. Likewise for reservoir clearing the complete forebay will be cleared as well as all areas below the reservoir limit which is considered to be the ground line defined as follows by HYDRO from the Full Supply Level of El. 192.0:

- a horizontal distance of 1 m for steep topography.
- 1 m along the bank slope for flat topography.

A drawing of the reservoir area (Dwg. 9.1) showing the types of vegetation contained within the reservoir limit has been prepared using aerial photographs and forestry maps. The affected area is estimated to be about 470 ha.

The provision of telecontrol equipment and transmission lines system are under HYDRO supply and management and their costs are not included in the Capital Cost Estimates.

Project direct costs were estimated for the following:

- construction of the works which are essential to the Development defined in this report;

- temporary facilities associated with the Development such as the camps, safety and security, first aid and ambulance services, warehousing, and the like.

Project indirect costs were allowed for on the following basis:

- Engineering and construction management was estimated at 12% of the direct cost. This cost covers tests and investigations, engineering studies and design, construction management, inspections and expediting, and commissioning of the equipment.

- Owner's cost is estimated at 5% of the direct cost. This allowance covers the management and legal costs which would be incurred by HYDRO.

- Contingency allowances were provided to cover presently unidentified problems and probable design changes. These costs were calculated as generally 10% of the civil works but increased to 15% where it is considered that inadequate information is available on subsurface conditions, 10% of the electro-mechanical works, 10% of the engineering and construction management, 10% of the owner's cost and 10% on the temporary facilities.

- Interest during construction was based on the projected cash flow of the capital cost of the Development at an internal rate of 12.683% per annum, which is equivalent to 3.030% on a quarterly basis and 1% on a monthly basis. The cash flow was dictated by the construction schedule and the projected commissioning date in 1991.

- Escalation was based on an allowance for civil works; electrical and mechanial equipment, temporary facilities, contingencies, engineering and construction management and Owner's costs at the rates detailed in Sub-Section 9.3 beginning July 1988 to the end of 1991.

Items which have been specifically excluded from these estimates are:

- Taxes, subsidies, royalties and custom duties. However, retail sales tax at 12% has been included where applicable.

- Land acquisition, resettlement and any measures associated with environmental or socio-economic remedies. Similarly, indirect benefits have been excluded.

- Cost of transmission line system beyond the powerhouse switchyard.

- Cost of telecontrol equipment.

The capital cost estimates, excluding allowances for escalation and interest during construction are shown in Table 9.1.

9.3 ESCALATION

Since the project is to be engineered and constructed over a period of 33 months, an allowance has been made for escalation during this period.

The annual rates used for escalating the project costs from 1988 to 1991 are assumed to be as follows:

1988/1989	5.02%
1989/1990	4.36%
1990/1991	4.01%
1991/1992	4.06%

9.4 PROJECT CASH FLOW

The estimated cash flow for the project in July 1988 terms and is shown as follows on a quarterly basis:

Year		Cash Flow in $ x 10^3
Year 1	1st Quarter	0
(1989)	2nd Quarter	836
	3rd Quarter	1,476
	4th Quarter	4,539
Year 2	1st Quarter	4,552
	2nd Quarter	12,374
	3rd Quarter	18,258
	4th Quarter	11,221
Year 3	1st Quarter	5,235
	2nd Quarter	7,512
	3rd Quarter	7,486
	4th Quarter	3,919

The project cash flow, including allowances for escalation and interest during construction is shown in Table 9.2 and is based on the construction program starting early in 1989 with completion scheduled for not later than December 1991, (YEAR 3) as shown in Fig. 8.1. This has been prepared on a quarterly basis and provides the data on which escalation and interest during construction is computed. These costs will vary with the overall rates for escalation and interest during construction, if the rates used are different from those assumed, and if the starting or completion date for construction is changed.

TABLE 9.1

CAPITAL COST ESTIMATES

ITEM	DESCRIPTION	COST $ x 1000
	CIVIL WORKS	
1	Permanent Access Roads	1,670.0
2	Cofferdams and Dewatering	456.8
3	Reservoir Clearing	2,350.0
4	Causeways	5,264.6
5	Spillway	4,436.1
6	Fish Pass	1,455.1
7	Power Canal	1,277.0
8	Powerhouse Civil Works	6,905.3
9	Tailrace	210.3
10	Switchyard	431.6
11	Saddle Dams	2,712.5
	Civil Works Sub-Total	27,169.3
	ELECTRO-MECHANICAL WORKS	
12	Hydraulic Gates	5,398.6
13	Generating Plant	11,946.6
14	Miscellaneous Electrical and Mechanical Equipment	2,208.9
	Electro-Mechanical Works Sub-Total	19,554.1
15	Switchyard Equipment	451.0
16	Temporary Facilities	6,504.0
	Construction Cost Total	53,678.4
17	Engineering and Construction Management	6,441.4
18	Owner's Cost	3,377.0
19	Contingencies	6,372.8
	CAPITAL COSTS TOTAL (in July 1988 Dollars)	69,869.6

NEWFOUNDLAND AND LABRADOR HYDRO
ROUND POND HYDROELECTRIC DEVELOPMENT
ESTIMATED DISBURSEMENT SCHEDULE (CASH FLOW)

TABLE 9.2

ITEM	DESCRIPTION	1 9 8 9 1st	2nd	3rd	4th	1 9 9 0 1st	2nd	3rd	4th	1 9 9 1 1st	2nd	3rd	4th	TOTAL $*1,000
1.0	CIVIL WORKS													
1.1	PERMANENT ACCESS ROAD			417	1,252									1,669
1.2	TEMPORARY ACCESS ROAD BRIDGES				440	110								550
1.3	ROAD MAINTENANCE						9	9	9	9	9	9	6	60
1.4	CONSTRUCTION CAMP				915	1,143	1,143							3,201
1.5	CONSTRUCTION CAMP MAINTENANCE AND OPERATION				299	178	396	560	434	97	294	323	112	2,693
1.6	RESERVOIR CLEARING							118	470	587	587	470	118	2,350
1.7	SPILLWAY DAM				80	183	1,173	3,233	61					4,730
1.8	CAUSEWAY DAMS						620	1,782	1,543		587	894		5,426
1.9	SADDLE DAMS						648	1,016	1,047					2,711
1.10	POWER CANAL						355	466	456					1,277
1.11	POWERHOUSE AND RETAINING WALLS						1,703	2,285	977	287	977	676		6,905
1.12	FISH PASSAGE FACILITY						85	904	417		49			1,455
1.13	TAILRACE											210		210
1.14	SWITCHYARD										322	110		432
2.0	ELECTRO - MECHANICAL WORKS													
2.1	HYDRAULIC GATES, STOPLOGS AND TRASHRACKS					439	1,642	2,013	854	241	159	51		5,399
2.2	GENERATING PLANT AND ASSOCIATED EQUIPMENT					902	1,562	1,644	1,644	1,644	1,644	1,453	1,453	11,946
2.3	MISCELLANEOUS ELECTRICAL AND MECHANICAL EQUIPMENT								325	488	488	844	517	2,662
3.0	TOTAL CONSTRUCTION COSTS			417	2,986	2,955	9,336	14,030	8,237	3,353	5,116	5,040	2,206	53,676
4.0	ENGINEERING AND MANAGEMENT		586	586	586	586	586	586	586	586	586	586	586	6,446
5.0	OWNER'S COSTS		142	269	297	297	367	419	355	301	321	320	289	3,377
6.0	SUB-TOTAL CAPITAL COSTS		728	1,272	3,869	3,838	10,289	15,035	9,178	4,240	6,023	5,946	3,081	63,499
7.0	CONTINGENCIES		72	126	383	380	1,055	1,541	908	419	596	588	305	6,373
8.0	TOTAL CAPITAL COSTS (1988)		800	1,398	4,252	4,218	11,344	16,576	10,086	4,659	6,619	6,534	3,386	69,872
9.0	QUARTERLY		800	1,398	4,252	4,218	11,344	16,576	10,086	4,659	6,619	6,534	3,386	69,872
10.0	ANNUALLY				6,450				42,224				21,198	69,872
11.0	ESCALATION		36	78	287	334	1,030	1,682	1,135	576	893	952	533	7,536
12.0	INTEREST DURING CONSTRUCTION		13	48	139	277	533	997	1,444	1,693	1,886	2,113	2,286	11,428
13.0	TOTAL QUARTERLY		848	1,524	4,678	4,828	12,907	19,255	12,665	6,928	9,398	9,600	6,205	88,836
	ANNUALLY				7,050				49,655				32,131	88,836

NOTES:

1. ESCALATION ASSUMED AS FOLLOWS:
   .1 BASE POINT JULY 1988
   .2 1988/1989 5.02%
   .3 1989/1990 4.36%
   .4 1990/1991 4.01%
   .5 1991/1992 4.06%

2. RATES FOR INTEREST DURING CONSTRUCTION ASSUMED AS FOLLOWS:
   .1 ANNUALLY 12.683%
   .2 QUARTERLY 3.030%
   .3 MONTHLY 1.000%

10. CONCLUSIONS AND RECOMMENDATIONS

10. CONCLUSIONS AND RECOMMENDATIONS

10.1 CONCLUSIONS

The following are the principal conclusions of the feasibility study:

1) It is fully feasible to construct the Development using the layout and procedures discussed in this report.

2) The total capital cost of the project excluding IDC and escalation is \$70 x 10^6 based on July 1988 prices.

   This cost does not include transmission beyond the switchyard, telecontrol, and environmental considerations.

3) An installation of 18 MW, a full supply level of 192.0 and a turbine design flow of 193 m^3/s have been selected for the development based on comparison of energy costs over a range of these parameters.

4) A Pit unit is the most economic type for the Development and a single unit was found to be more economic than two units.

5) The construction schedule developed from the layouts and quantities, gives a total duration of construction of 33 months. To meet an assumed on-power date of December 15, 1991, engineering design would be required to commence on March 1, 1989 for the access road and on April 1, 1989 for preliminary engineering leading to tender documentation for the remainder of the Works.

10.2 RECOMMENDATIONS

Prior to final design the following work would be required:

- a complete survey of the access road alignment and stream crossings;

- a ground survey and subsurface investigation of the construction camp site;

- a study to determine the type, size and location of fish pass required for the development;

- a survey of the East Channel of the Round Pond outlet to enable the tailwater levels and discharge velocities to be determined for the Spillway.

- further investigation of a combined spillway/powerhouse arrangement in conjunction with structure alignment No. 3 (i.e. downstream of the confluence of the East and West Channels) to determine if there is a significant cost advantage.

REFERENCE LIST

REFERENCE LIST

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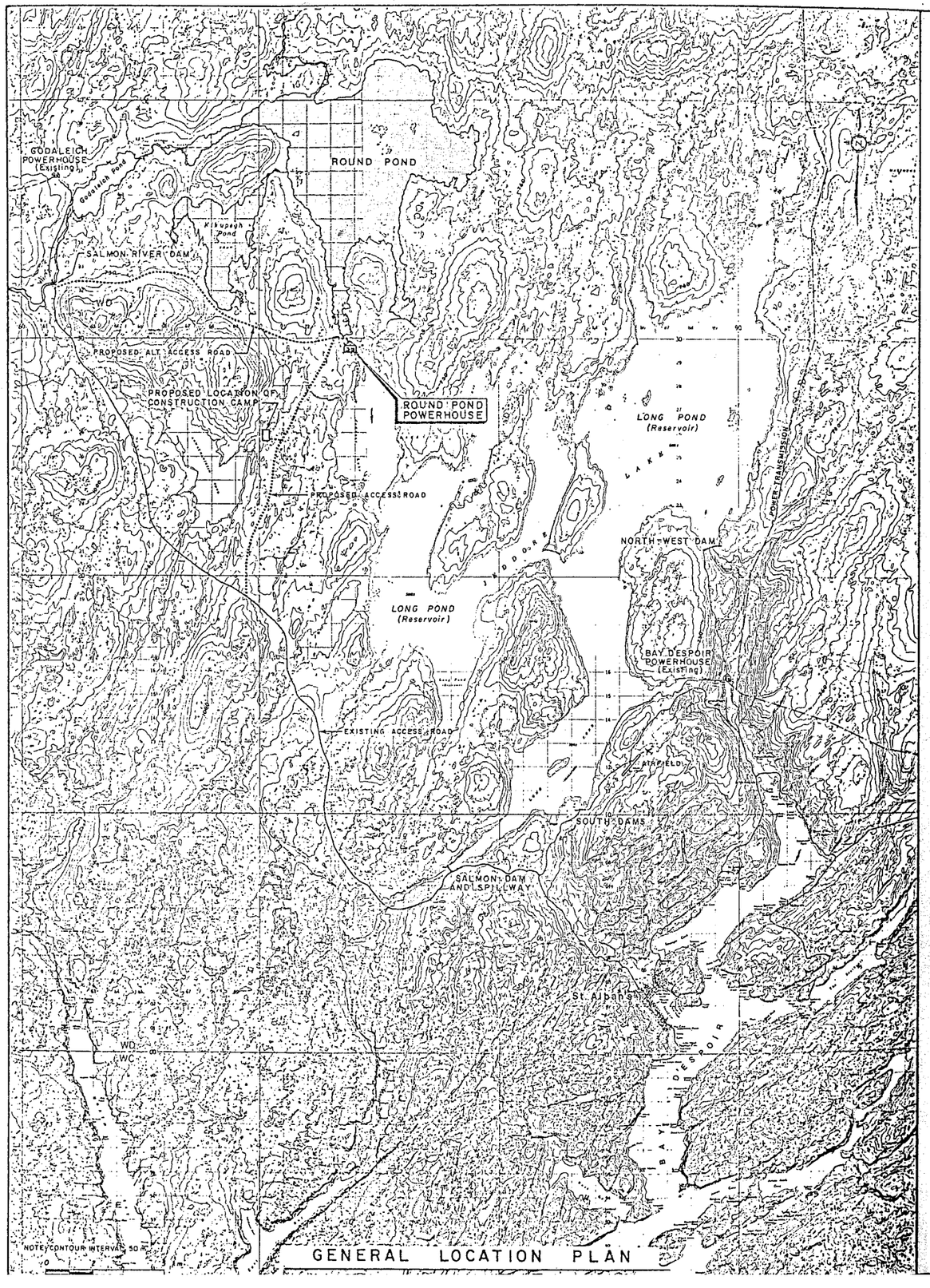
(3) T. Saville, E.W. McClendon & A.L. Cochrane. Freeboard Allowances for Waves in Inland Reservoirs.

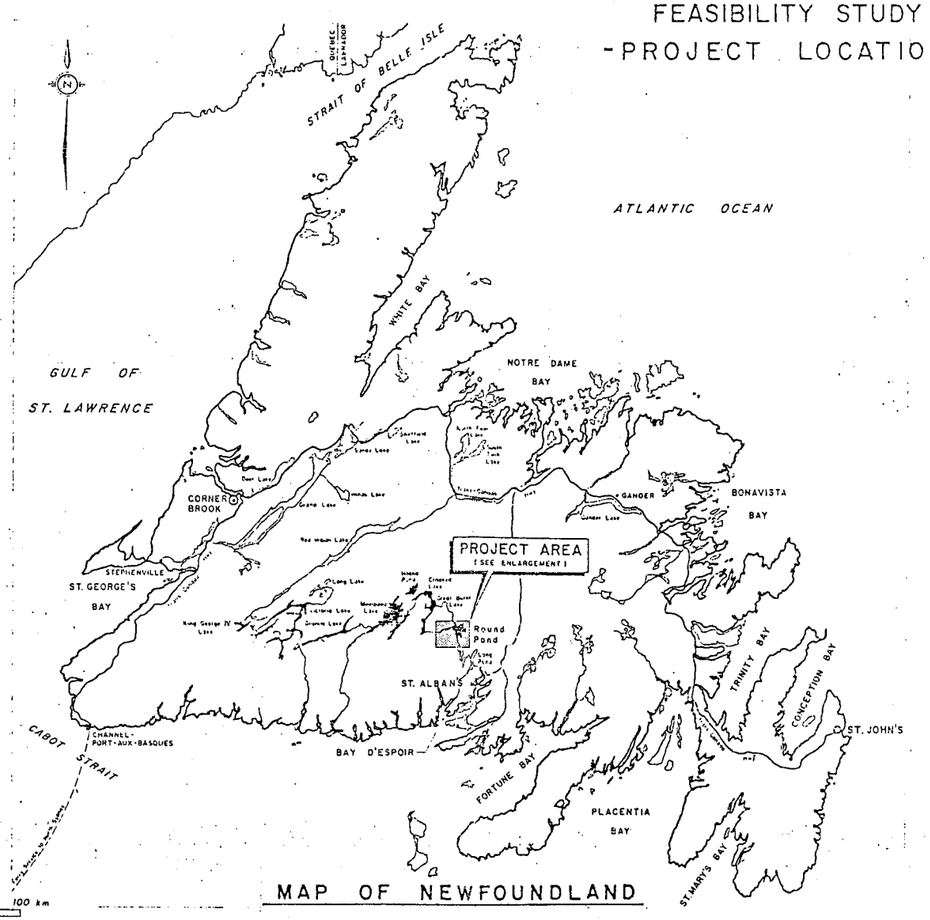
(4) U.S. Army Corps of Engineers - Hydraulic Design of Spillways, Plate 31.

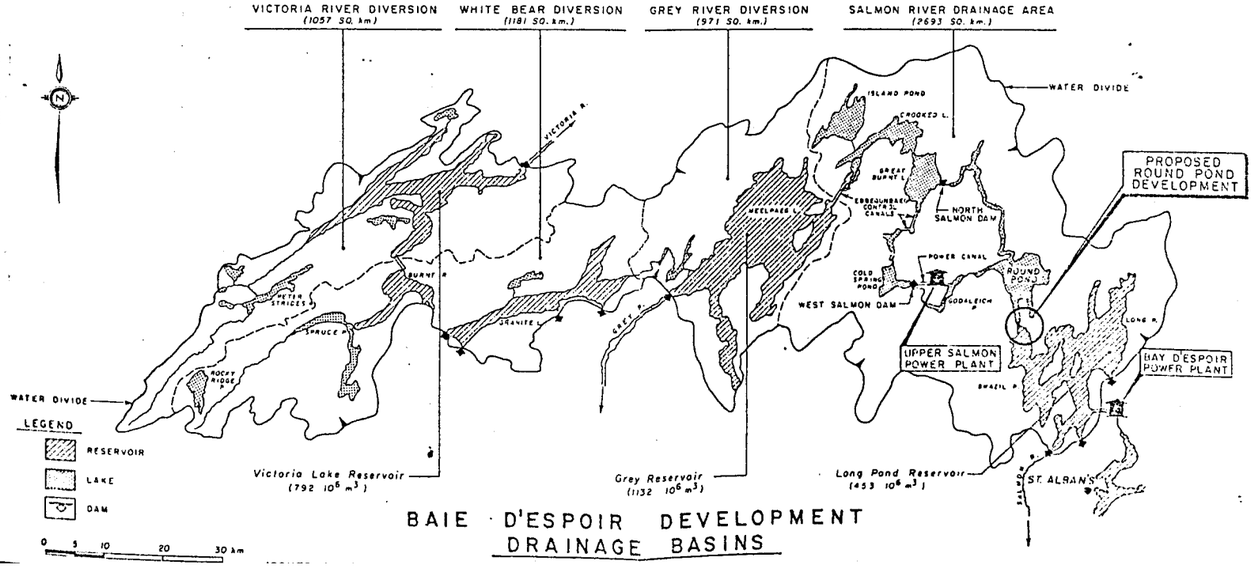
DRAWING 2.1

ROUND POND HYDROELECTRIC DEVELOPMENT FEASIBILITY STUDY

-PROJECT LOCATION-







DRAWING 1

ROUND POND HYDROELECTRIC
FEASIBILITY
- TERRAIN ANALYSIS
(FROM AERIAL PHOTO INTERP

KIKUPEGH POND

D ROUND POND

APPROXIMATE LOCATION OF EXISTING ROAD

UPPER SALMON ROAD

FREQUENT LARGE BOULDERS OVER ENTIRE SURFACE

0,45 m COVER OVER TILL

0,3 m COVER OVER TILL

0,4 m COVER OVER TILL

> 3,0 m PEAT

1,1 m PEAT

0,25 m PEAT

0,25 m COVER OVER TILL

0,25-0,50 m PEAT

0,3 m COVER OVER TILL

ALTERNATIVE B

ALTERNATIVE A

PROJECT AREA

ST GEORGE'S BAY

GABBOT STRAIT

SALMON

LEGEND

- ⊠ — POTENTIAL BORROW AREAS
- ⊗ — EXPLORATION POINTS
- Tv — TILL VENEER 1-3 m DEEP
- Th — HUMMOCKY TILL 3 m DEEP
- Cx — COLLUVIAL COMPLEX SLOPES 2-3 m DEEP
- Tv/R — TILL VENEER OVER BEDROCK < 1 m DEEP
- Tr — RIDGED TILL 3 m DEEP
- B — BOG 1,5-2,0 m DEEP
- F — FEN 0,5-1,0 m DEEP
- Sm — STREAM MARSH 0,5-1,0 m DEEP
- = = — UPPER SALMON ROAD
- — — — PROPOSED ROUND POND ACCESS ROAD

SCALE 1:15,840

0 500 1000 1500 2000 m

ISSUED: JUNE 88, SEPT. 88

ROUND POND
W.L. 186.0 m ±

AXIS OF STRUCTURES

BH 8
BH 7
BH 6
BH 5
BH 4
BH 3
BH 2
BH 1

BM 1
BM 2
BM 3
BM 4

T3-3
T3-2
T3-1

WEST CAUSEWAY
CENTRAL CAUSEWAY
SPILLWAY
EAST CAUSEWAY
POWERHOUSE

AXIS OF INTAKE CANAL AND POWERHOUSE

A (DWG. 6.3)

SADDLE DAM No. 2
SADDLE DAM No. 1

LONG POND RESERVOIR
N.T.L. 180.75 m

STRING BOG

BLDG

W.L. 200.3 ±
W.L. 190.9 ±
W.L. 184.1
W.L. 184.4

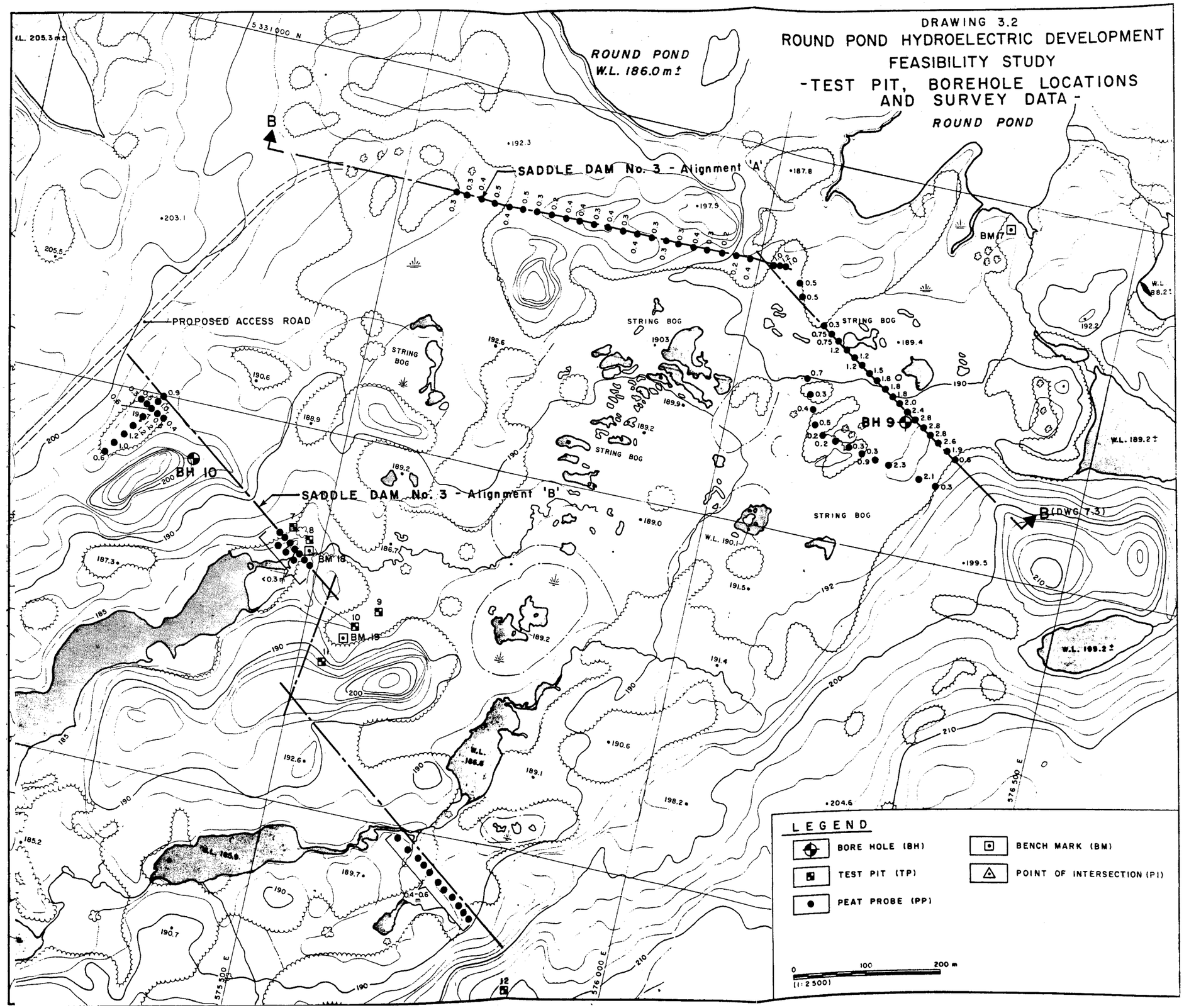
5 330 500 N
5 330 000 N
5 329 500 N
574 500 E
575 000 E

ISSUED: JUNE 88, SEPT 88 (1)

DRAWING 3.2

ROUND POND HYDROELECTRIC DEVELOPMENT FEASIBILITY STUDY

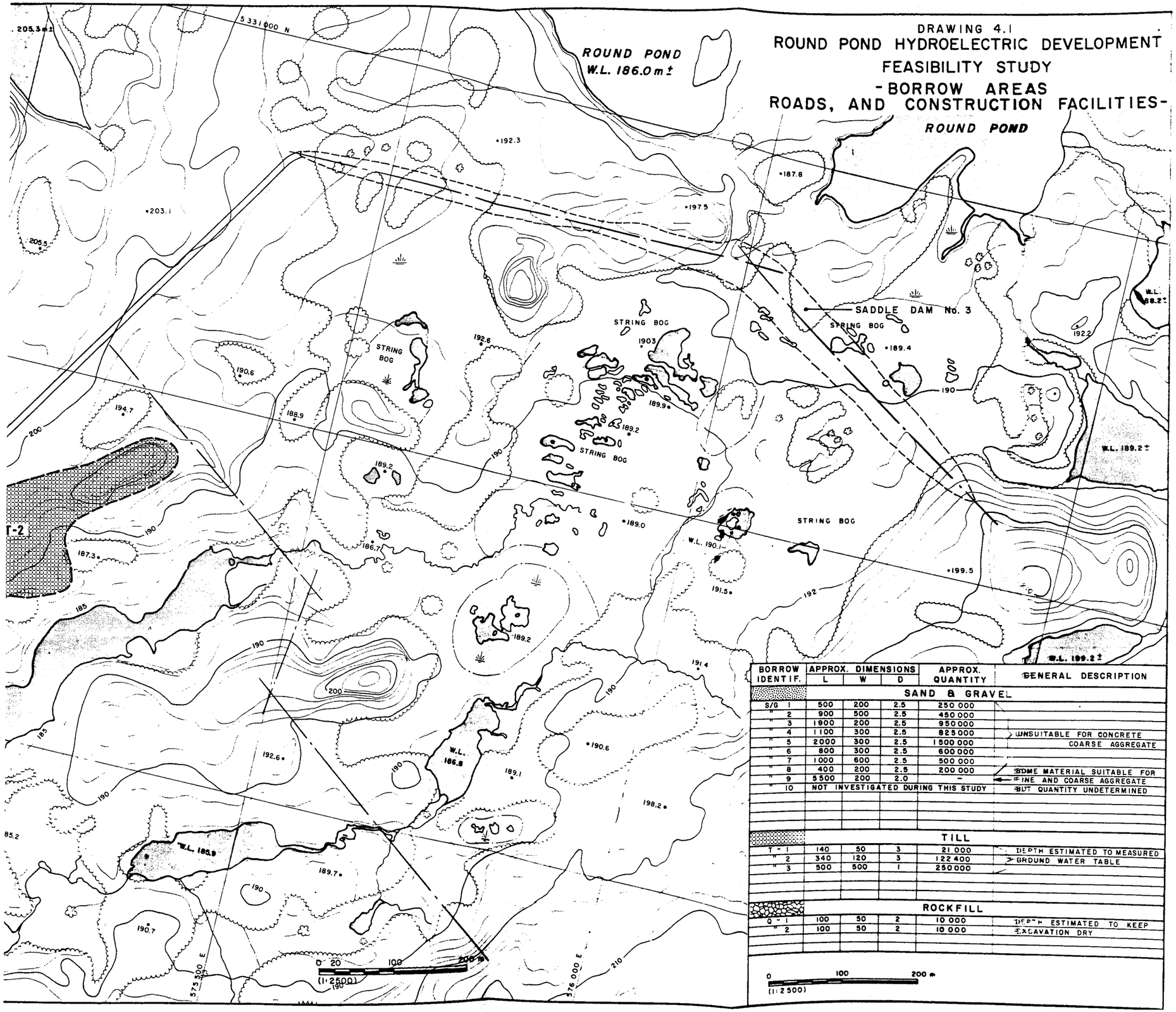
-TEST PIT, BOREHOLE LOCATIONS AND SURVEY DATA-



ROUND POND
W.L. 186.0m ±
T-3
COFFERDAM No. 2
COFFERDAM No. 1
Q-2
Q-1
WEST CAUSEWAY
CENTRAL CAUSEWAY
EAST CAUSEWAY
SPILLWAY
INTAKE POWER CHANNEL
SADDLE DAMS 1 & 2
T-1
POWERHOUSE
LONG POND RESERVOIR
NTL 180.75 m.
BOG
5 330 500 N
5 329 500 N
575 500 E
574 000 E
574 500 E
For Continuation See Dwg. 2.1
PROPOSED ALT. ACCESS ROAD
POWERHOUSE
S/G 1
S/G 2
S/G 3
S/G 4
S/G 5
S/G 6
S/G 7
S/G 8
S/G 9
S/G 10
PROPOSED CONSTRUCTION CAMP
PROPOSED ACCESS ROAD
NOEL PAULS ESKER
EXISTING SALMON ROAD
0 1.0 2.0 km.
W.L. 190.9 ±
W.L. 184.1
BLDG.
STRING BOG
ISSUED: JUNE/88, SEPT. 88 (1)

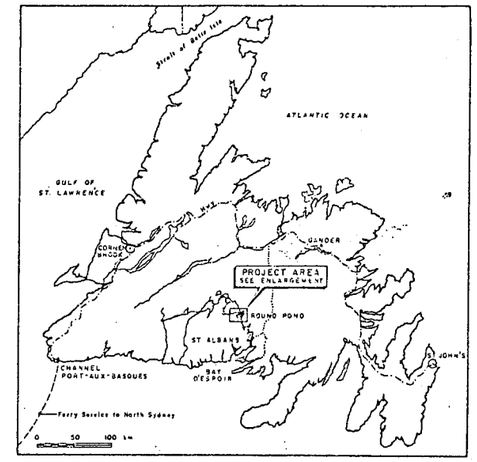
DRAWING 4.1

ROUND POND HYDROELECTRIC DEVELOPMENT FEASIBILITY STUDY - BORROW AREAS ROADS, AND CONSTRUCTION FACILITIES -

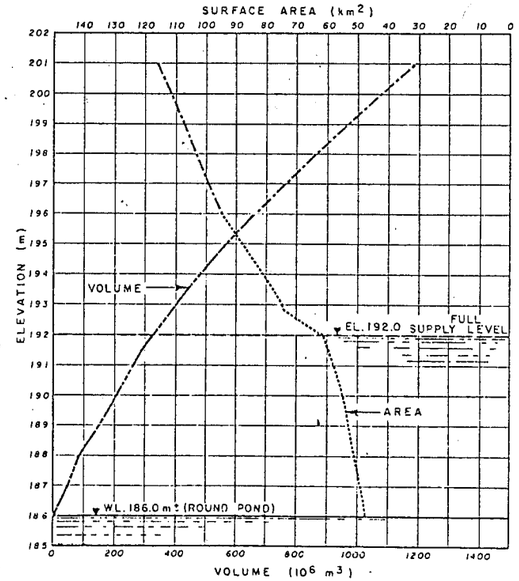


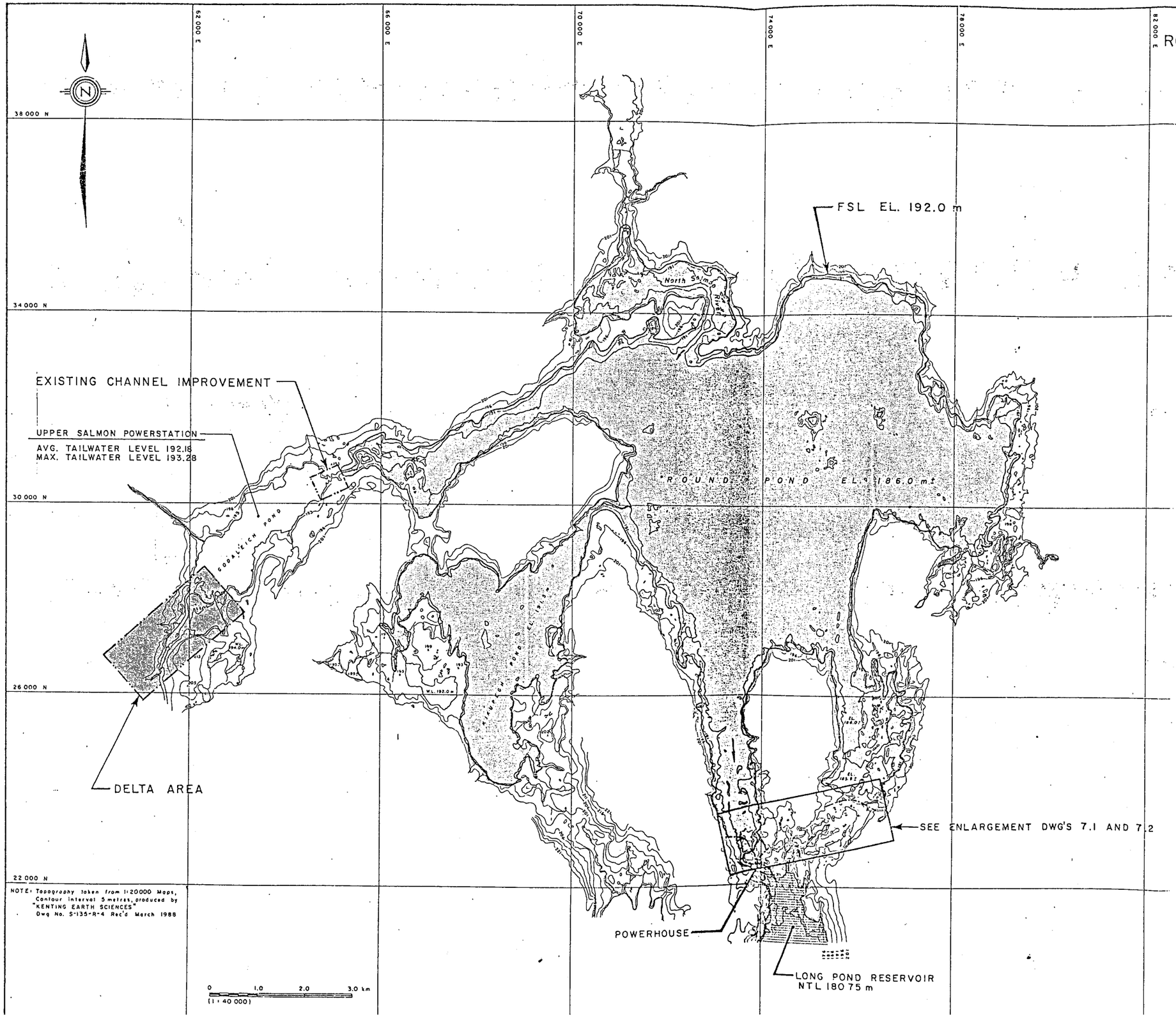
BORROW IDENTIF.	APPROX. DIMENSIONS L	W	D	APPROX. QUANTITY	GENERAL DESCRIPTION
SAND & GRAVEL					
S/G 1	500	200	2.5	250 000	
" 2	900	500	2.5	450 000	
" 3	1900	200	2.5	950 000	
" 4	1100	300	2.5	825 000	UNSUITABLE FOR CONCRETE COARSE AGGREGATE
" 5	2000	300	2.5	1 500 000	
" 6	800	300	2.5	600 000	
" 7	1000	600	2.5	500 000	
" 8	400	200	2.5	200 000	SOME MATERIAL SUITABLE FOR FINE AND COARSE AGGREGATE
" 9	5500	200	2.0	-	BUT QUANTITY UNDETERMINED
" 10	NOT INVESTIGATED DURING THIS STUDY				
TILL					
T - 1	140	50	3	21 000	DEPTH ESTIMATED TO MEASURED GROUND WATER TABLE
" 2	340	120	3	122 400	
" 3	500	500	1	250 000	
ROCKFILL					
Q - 1	100	50	2	10 000	DEPTH ESTIMATED TO KEEP EXCAVATION DRY
" 2	100	50	2	10 000	

DRAWING 6.1
ROUND POND HYDROELECTRIC DEVELOPMENT
FEASIBILITY STUDY
- RESERVOIR -



MAP OF NEWFOUNDLAND





NOTE: Topography taken from 1:20000 Maps, Contour Interval 5 metres, produced by "KENTING EARTH SCIENCES" Dwg No. S-135-R-4 Rec'd March 1988

ROUND POND
W.L. 186.0 m ±

ROUND POND RESERVOIR FSL 192.0

W.L. 205.3 m ±

STRING BOG

STRING BOG

W.L. 200.3 ±

W.L. 190.9 ±

W.L. 184.1

W.L. 184.1

SADDLE DAM ACC

WEST CAUSEWAY

CENTRAL CAUSEWAY

EAST CAUSEWAY

SPILLWAY

POWER INTAKE CANAL

SADDLE DAM No. 2

SADDLE DAM No. 1

POWERHOUSE

TAILRACE CANAL

SWITCHYARD

LONG POND RESERVOIR
NTL. 180.75 m

BLDG.

5 330 500 N

5 330 000 N

5 329 500 N

573 500 E

574 000 E

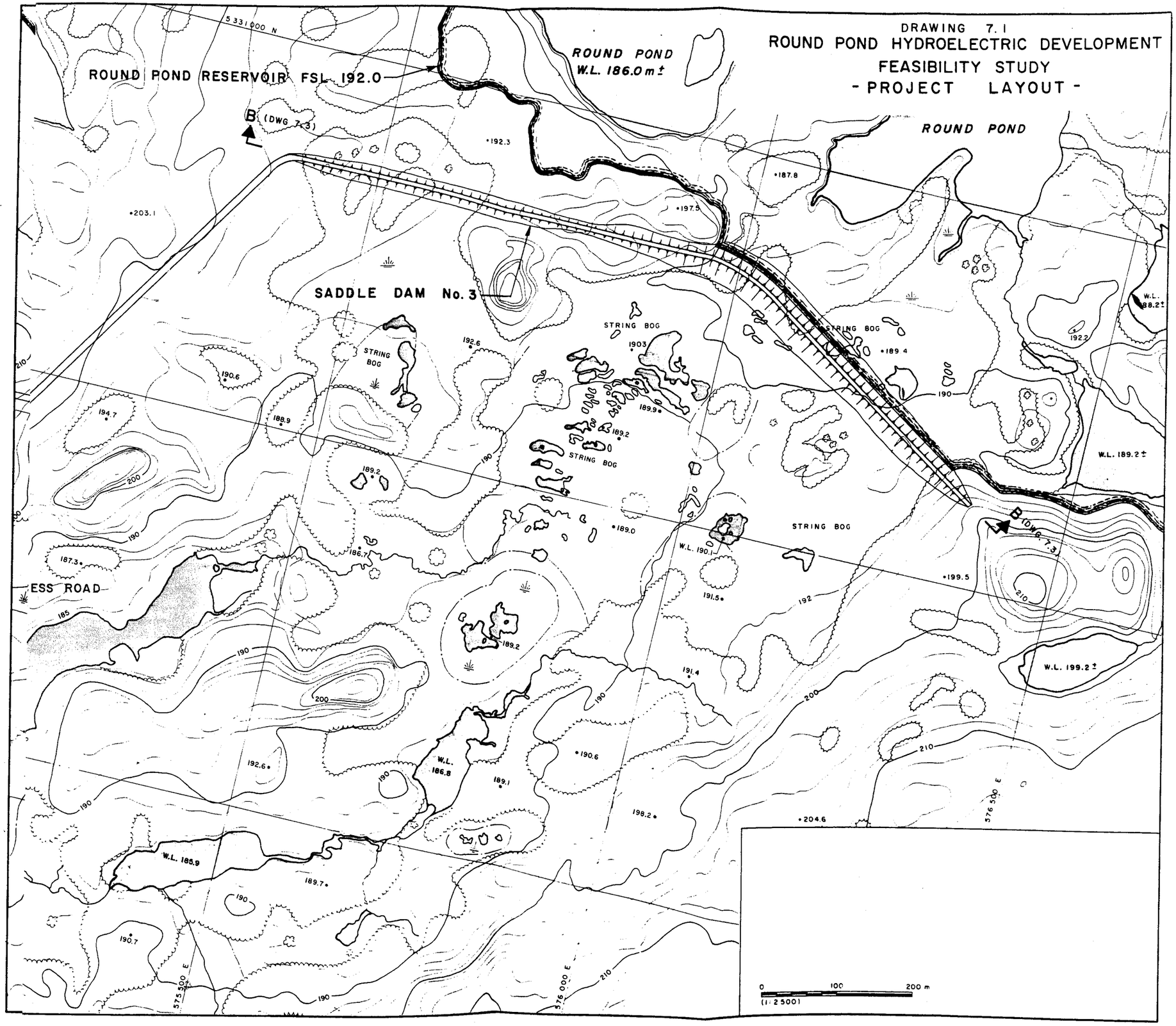
574 500 E

575 000 E

ISSUED: JUNE 88, SEPT. 88

DRAWING 7.1

ROUND POND HYDROELECTRIC DEVELOPMENT
FEASIBILITY STUDY
- PROJECT LAYOUT -



DRAWING 7.2

ROUND POND HYDROELECTRIC DEVE
FEASIBILITY STUDY
- STRUCTURE LAYOUT -

ROUND POND RESERVOIR
FSL 192.0 m

FSL 192.0 m

COFFERDAM No. 1

COFFERDAM No. 2

573 500 E

574 000 E

TO SADDLE DAM No.3 1.5 km±

A (DWG 7.3)

D (DWG 7.3)

SADDLE DAM No. 2

INTAKE CANAL
(DWG 7.7) G

E (DWG 7.7)

SPILLWAY
(DWG 7.4)

STRIP TO BEDROCK

CENTRAL CAUSEWAY

WEST CAUSEWAY

C (DWG 7.3)

BRIDGE
(1-25 m SPAN)

BRIDGE
(2-25 m SPANS)
ROCK FILLED TIMBER CRIBS

EAST CAUSEWAY

F (DWG 7.7)

EL. 196.5

SLOPE ROAD 6.7% Gr.

TEMPORARY CONSTRUCTION ROAD

TO UPPER SALMON ROAD 12 km±

5 330 000 N

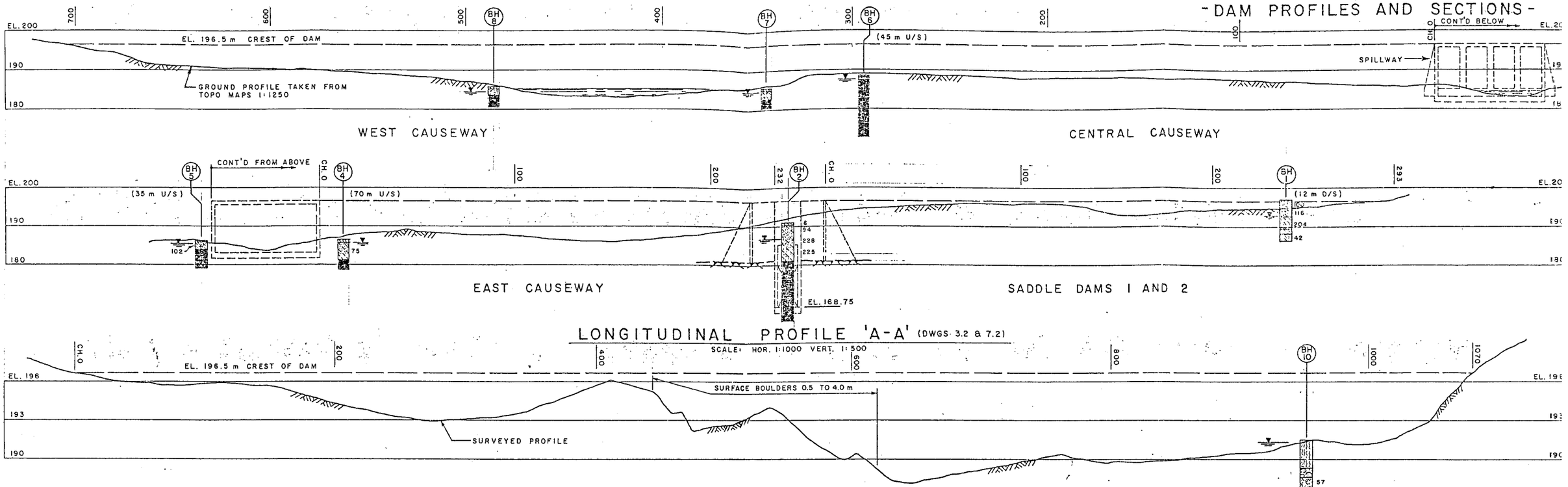
329 500 N

CH. 0
CH. 100
CH. 200
CH. 293
CH. 300
CH. 400
CH. 500
CH. 600
CH. 700
CH. 720
CH. 285

186 187 188 189 190 192 194 196 200

DRAWING 7.3

ROUND POND HYDROELECTRIC DEVELOPMENT
FEASIBILITY STUDY
- DAM PROFILES AND SECTIONS -

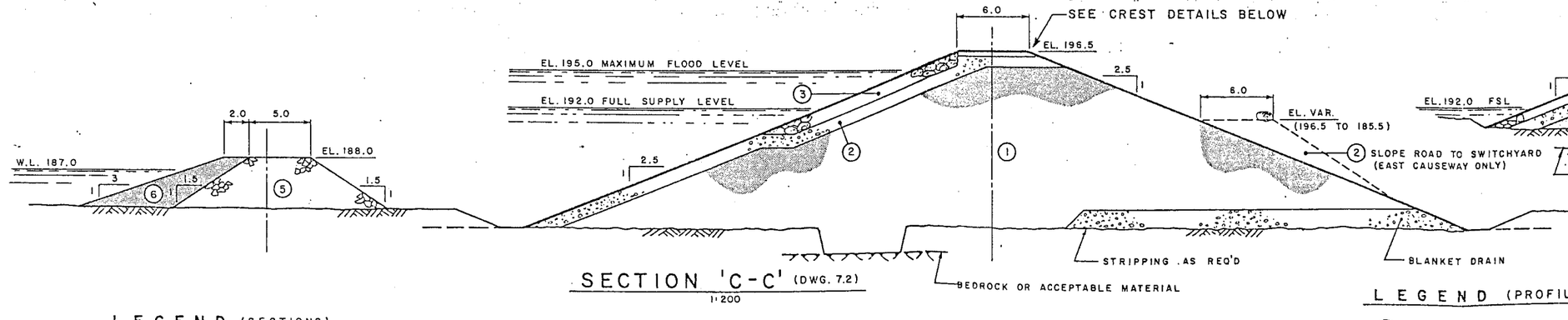


LONGITUDINAL PROFILE 'A-A' (DWGS 3.2 & 7.2)

SCALE: HOR. 1:1000 VERT. 1:500

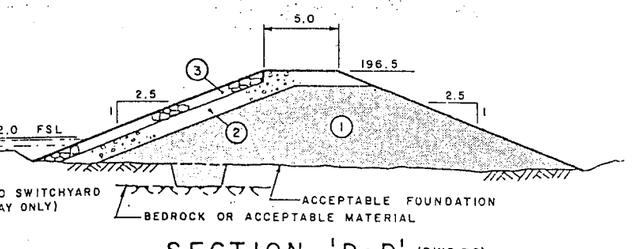
LONGITUDINAL PROFILE 'B-B' (DWGS 3.2 & 7.1)

SCALE: HOR. 1:1500 VERT. 1:150



SECTION 'C-C' (DWG. 7.2)

1:200

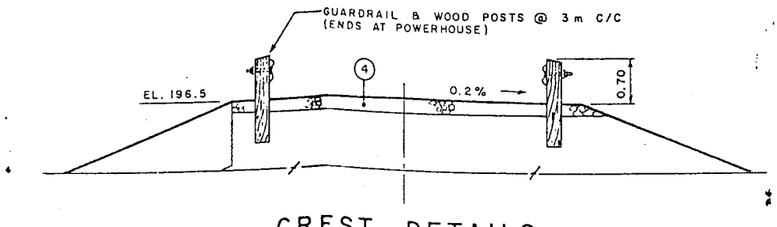


SECTION 'D-D' (DWG. 7.2)

1:200

LEGEND (SECTIONS)

1. ROLLED IMPERVIOUS FILL (TILL)
2. TRANSITION
3. RIPRAP
4. GRANULAR SURFACING
5. DUMPED ROCKFILL
6. DUMPED IMPERVIOUS



CREST DETAILS

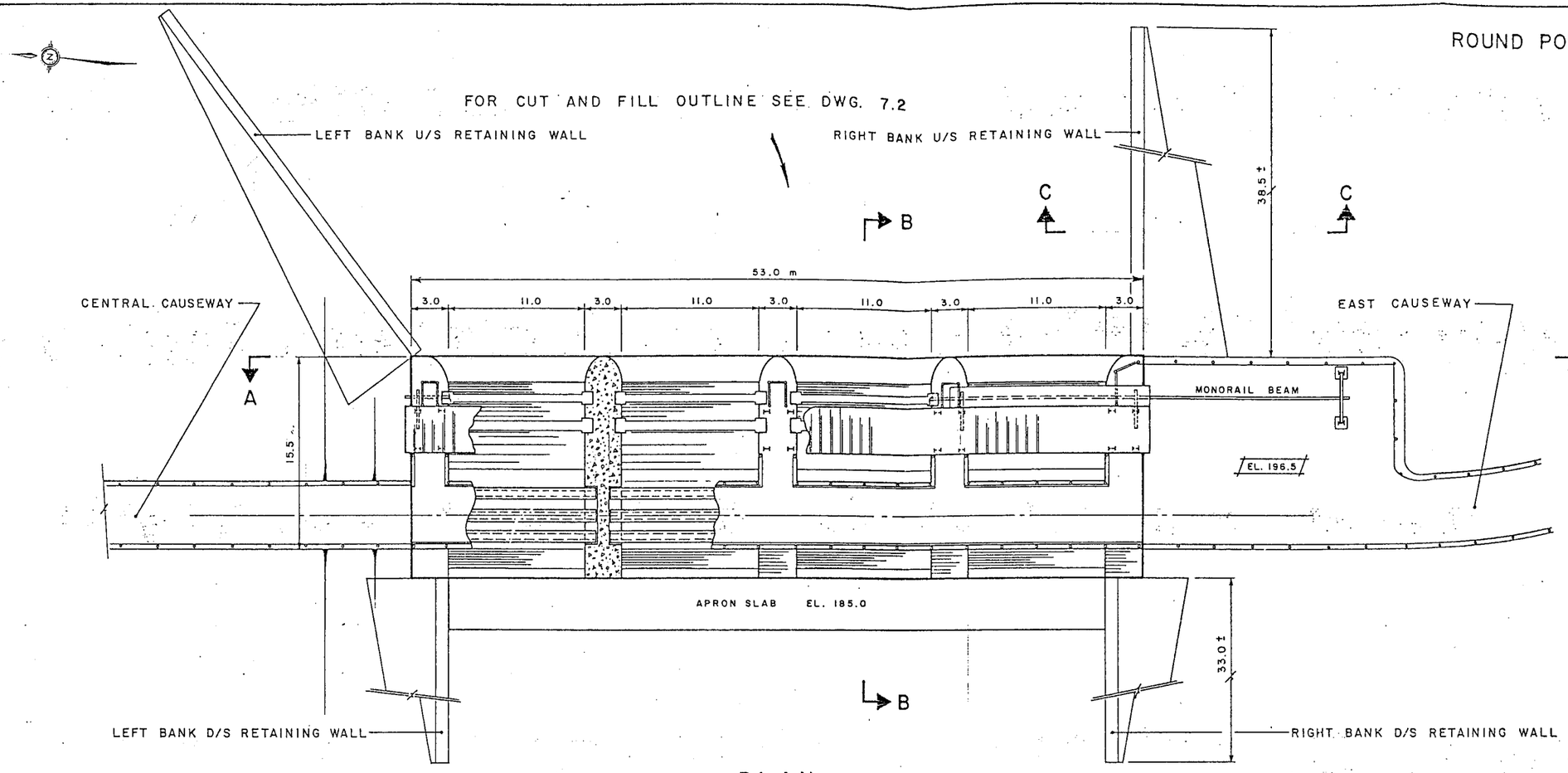
LEGEND (PROFILES)

- BH 2 — BOREHOLE IDENTIFICATION
- (35 m U/S) — BOREHOLE LOCATED 35 m UPSTREAM OF SECTION
- 6 — STANDARD PENETRATION RESISTANCE 'N' VALUE - BLOWS/0.3 m
- WATER LEVEL
- ROOTMAT
- SANDY SILT TO SILT
- SILTY SAND AND GRAVEL
- SAND AND GRAVEL
- BEDROCK (SILTSTONE AND SANDSTONE)

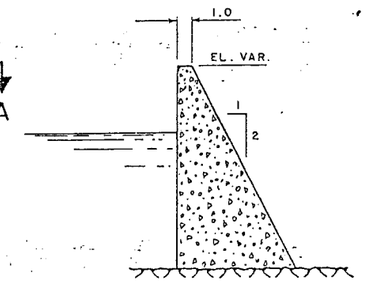
0	2.5	5.0 (1:50)
0	7.5	15.0 (1:150)
0	10.0	20.0 (1:200)
0	25.0	50.0 (1:500)
0	50.0	100.0 (1:1000)
0	75.0	150.0 (1:1500)

DRAWING 7.4

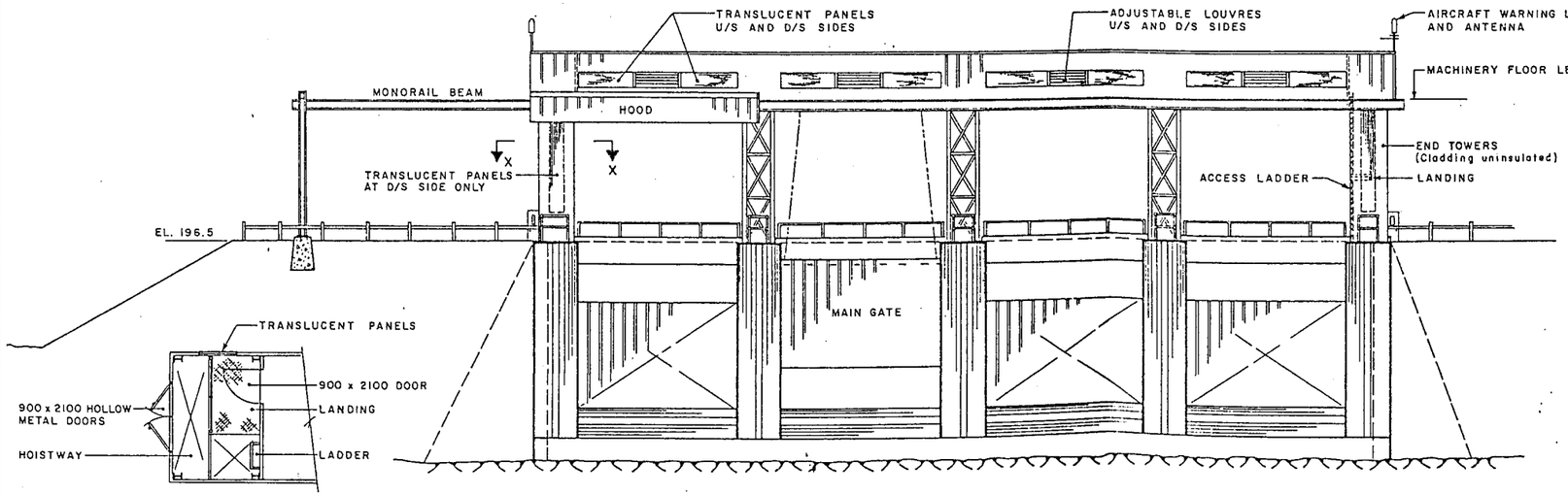
ROUND POND HYDROELECTRIC DEVELOPMENT
FEASIBILITY STUDY
- SPILLWAY -



PLAN
1:200

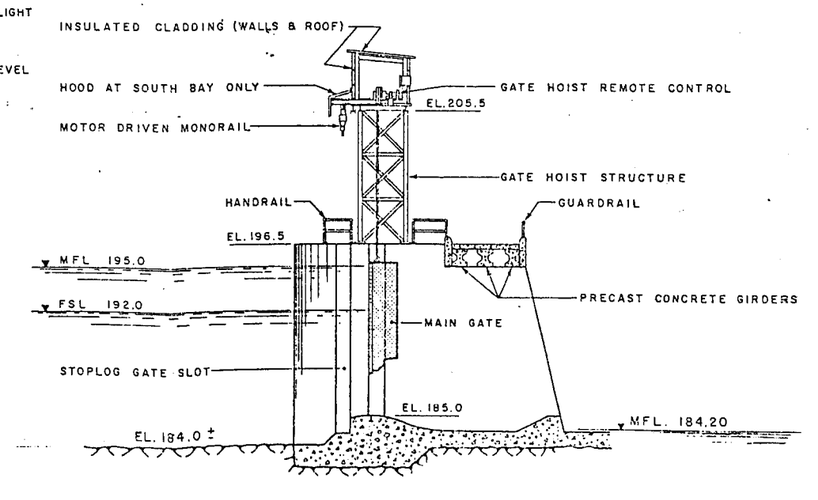


SECTION 'C-C'
1:200



SECTIONAL PLAN 'X-X'
(NORTH TOWER OPPOSITE HAND)

ELEVATION 'A-A'



SECTION 'B-B'

0 10 20 (1:200)

DRAWING 7.5

ROUND POND HYDROELECTRIC DEVELOPMENT FEASIBILITY STUDY

- GENERAL ARRANGEMENT OF POWERHOUSE AREA

N

TOE OF DAM

B (DWG. 7.6)

5 329 500 N

TOE OF DAM

LIMIT OF EXCAVATION

U/S RETAINING WALL

D/S RETAINING WALL

BERM ON BEDROCK EL. 181.0 ±

I.E. 190.3

BERM ON BEDROCK EL. 181.0 ±

WALKWAY EL. 182.0 ±

I.E. 181.4

FISH PASSAGE WALKWAY-SLAB ON GRADE

BACKFILL TO UNDERSIDE FISH PASSAGE & GRADE AREA

ROADWAY DECK

MAN DOOR

BERM ON BEDROCK EL. 181.0 ±

POWER CANAL

22000

20 % UP TO EL. 179.5

EL. 168.75

12500

A

INTAKE

POWERHOUSE

A (DWG. 7.6)

EL 171.0

14000

TAILRACE CANAL

FISH SCREEN

FISH SCREEN

20% UP

48000

OVERHEAD DOOR WITH MAN DOOR

EL. 196.5

5 x 100 ø PVC TO UNIT BREAKER & CONTROL

BERM ON BEDROCK EL. 181.0 ±

U/S RETAINING WALL

573 900 E

185.8

66/15 SWITCHYARD

SECURITY FENCE 17 x 24 m

573 900 E

LIMIT OF EXCAVATION

B

TOE OF DAM

PARKING LOT

185.6

RIPRAP

GUARD RAIL

EXISTING
LONG POND RESERVOIR
MFL 184.20 m
NTL 180.75 m

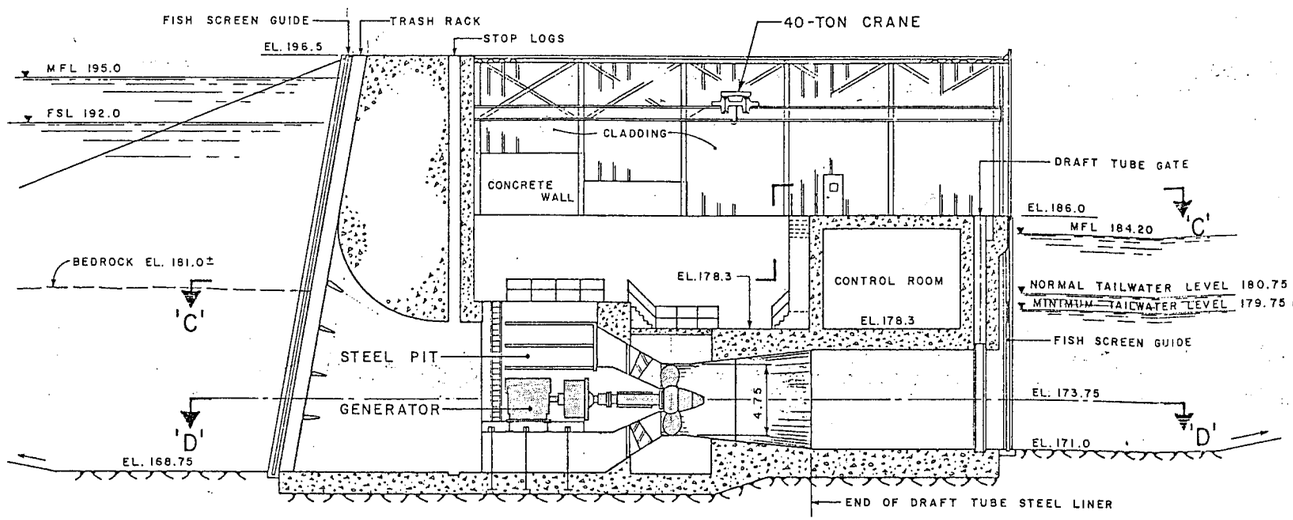
5 329 500 N

0 2 4 6 8 10 20 m
(1:200)

DRAWING 7.6

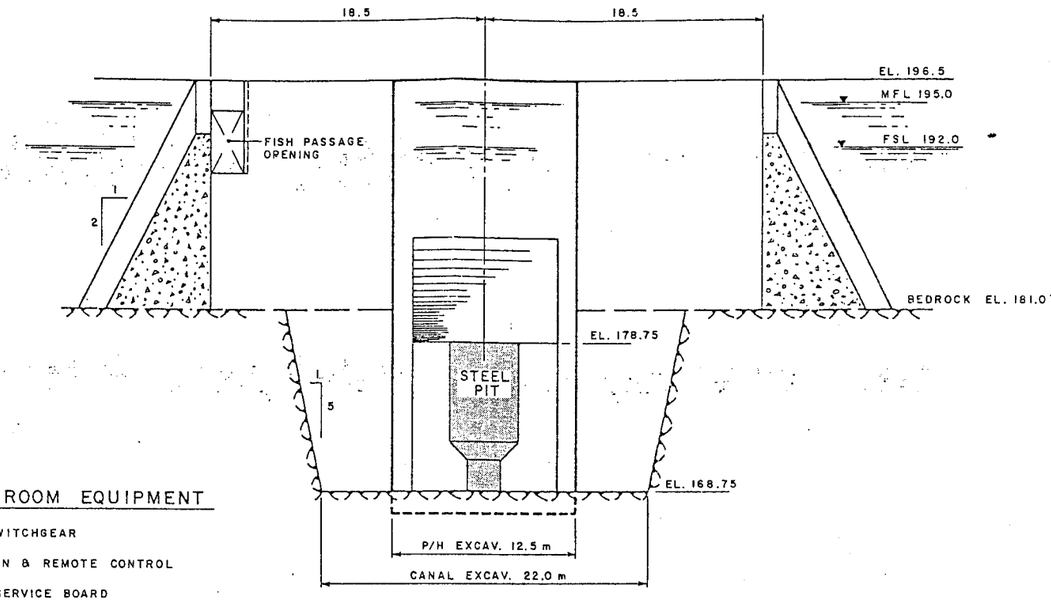
ROUND POND HYDROELECTRIC DEVELOPMENT FEASIBILITY STUDY

- POWERHOUSE -



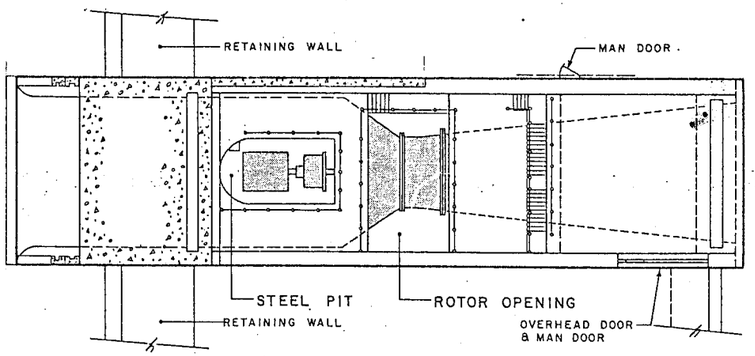
SECTION 'A - A' (DWG. 7.5)

1:200



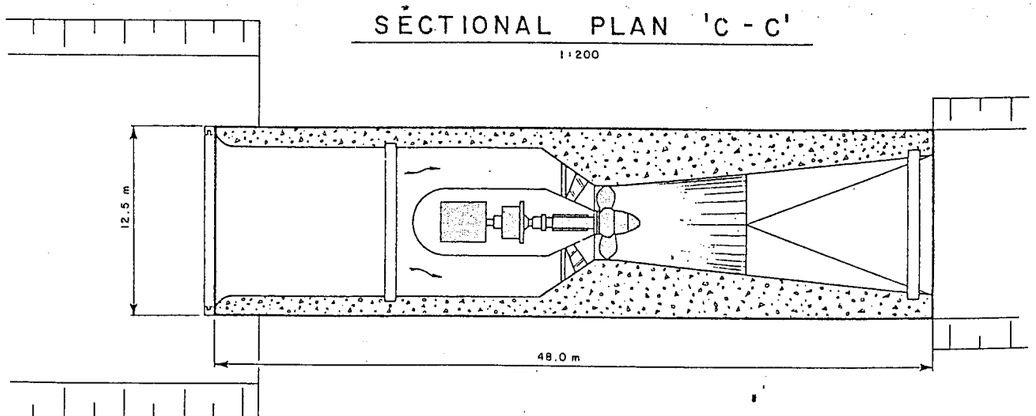
ELEVATION 'B - B' (DWG. - 7.5)

1:200



SECTIONAL PLAN 'C - C'

1:200

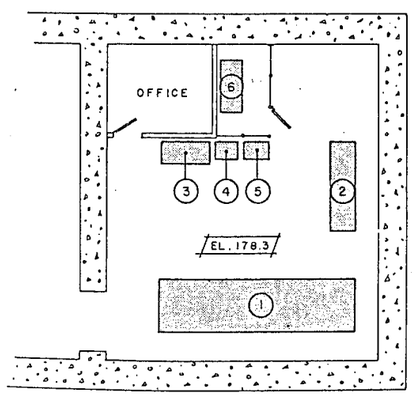


SECTIONAL PLAN 'D - D'

1:200

CONTROL ROOM EQUIPMENT

1. POWER SWITCHGEAR
2. PROTECTION & REMOTE CONTROL
3. STATION SERVICE BOARD
4. CHARGER
5. DC BOARD
6. STATION BATTERY



CONTROL ROOM LAYOUT

1:100

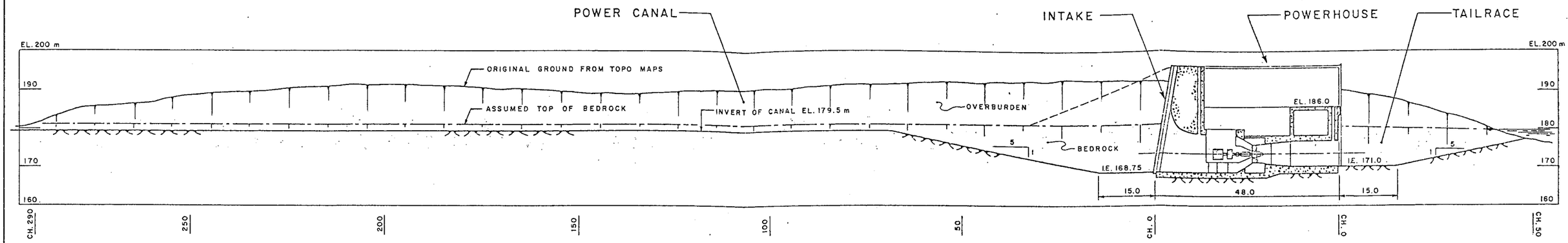
0 1 2 3 4 5
0 2 4 6 8 10
10 m (1:100)
20 m (1:200)

ISSUED: JUNE 88, SEPT. 88 (1)

DRAWING 7.7

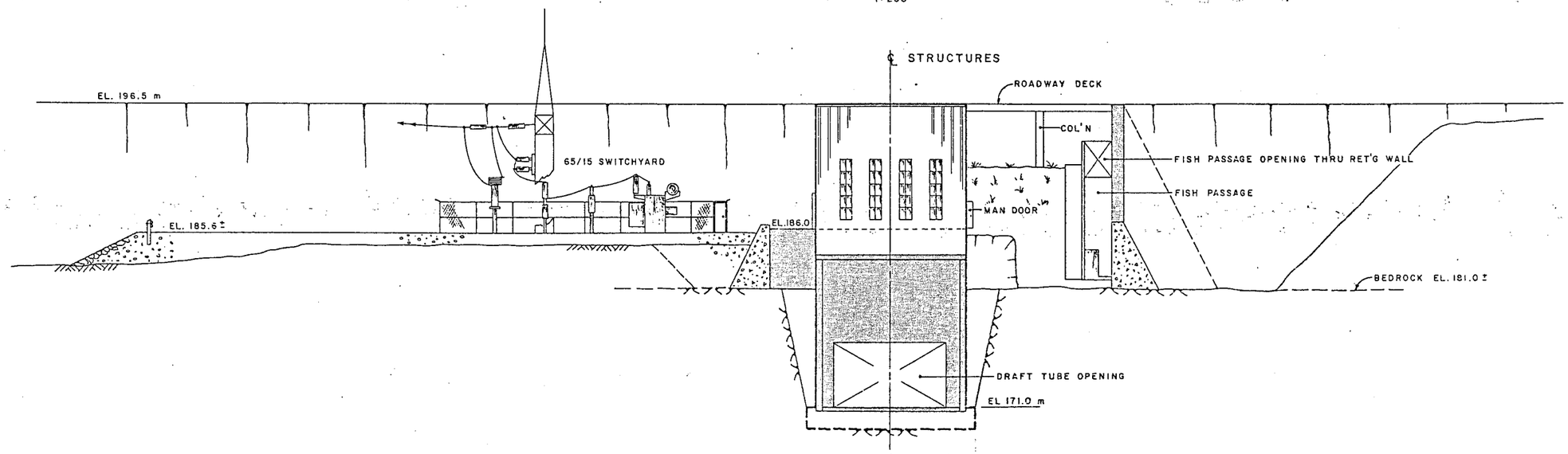
ROUND POND HYDROELECTRIC DEVELOPMENT FEASIBILITY STUDY

- POWER CANAL - -LONGITUDINAL PROFILE AND SECTIONS-



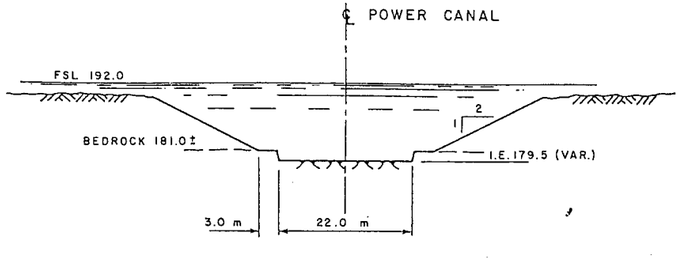
LONGITUDINAL PROFILE 'E-E' (DWG 7.2)

1:200



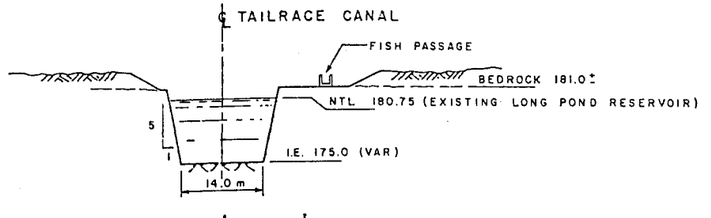
SECTION 'F-F' (DWG 7.2)

1:200



SECTION 'G-G' (DWG. 7.2)

1:500



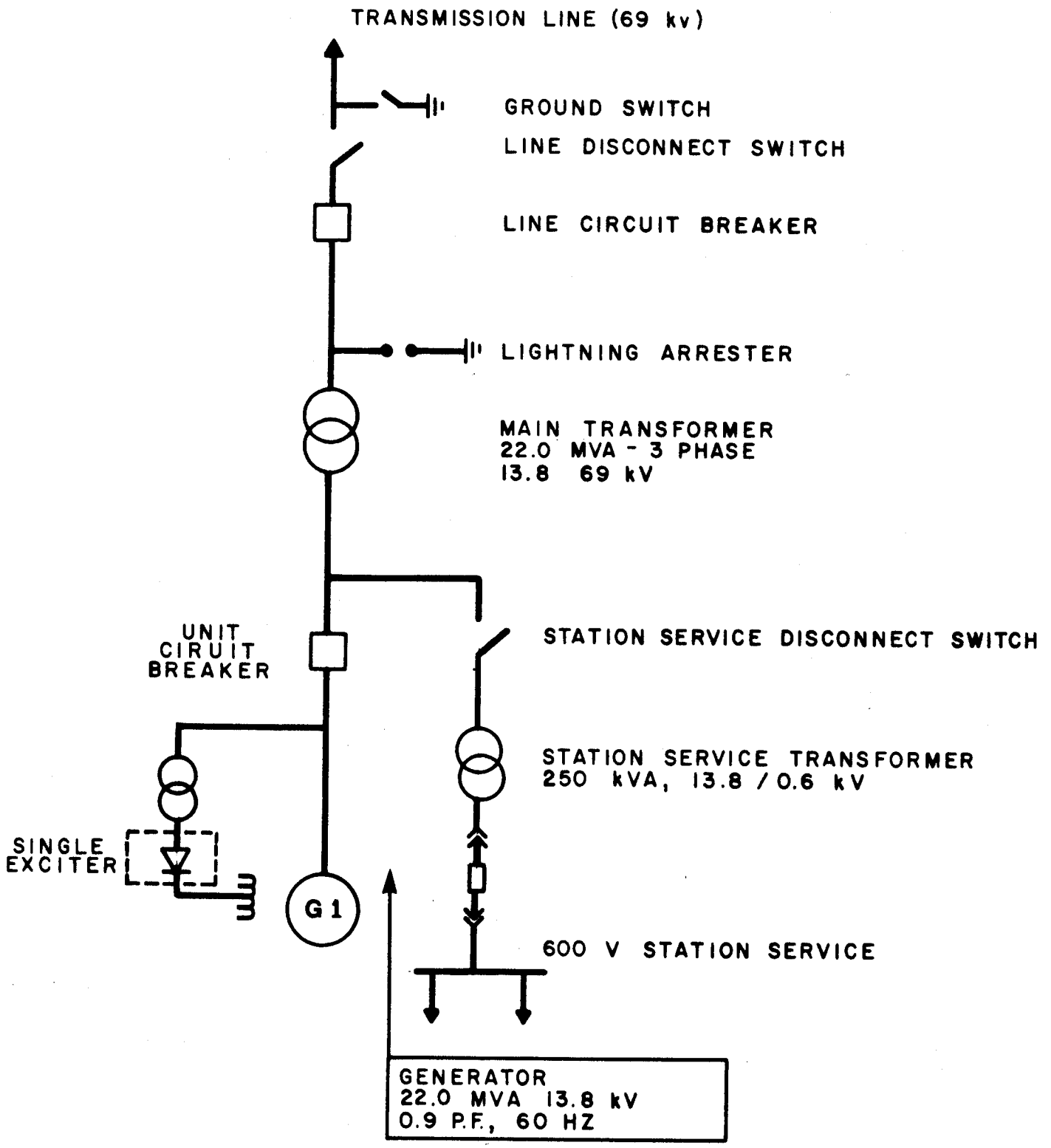
SECTION 'H-H' (DWG 7.2)

1:500

0 10 20 m (1:200)
0 25 50 m (1:500)

ISSUED: JUNE 88 SEPT 88

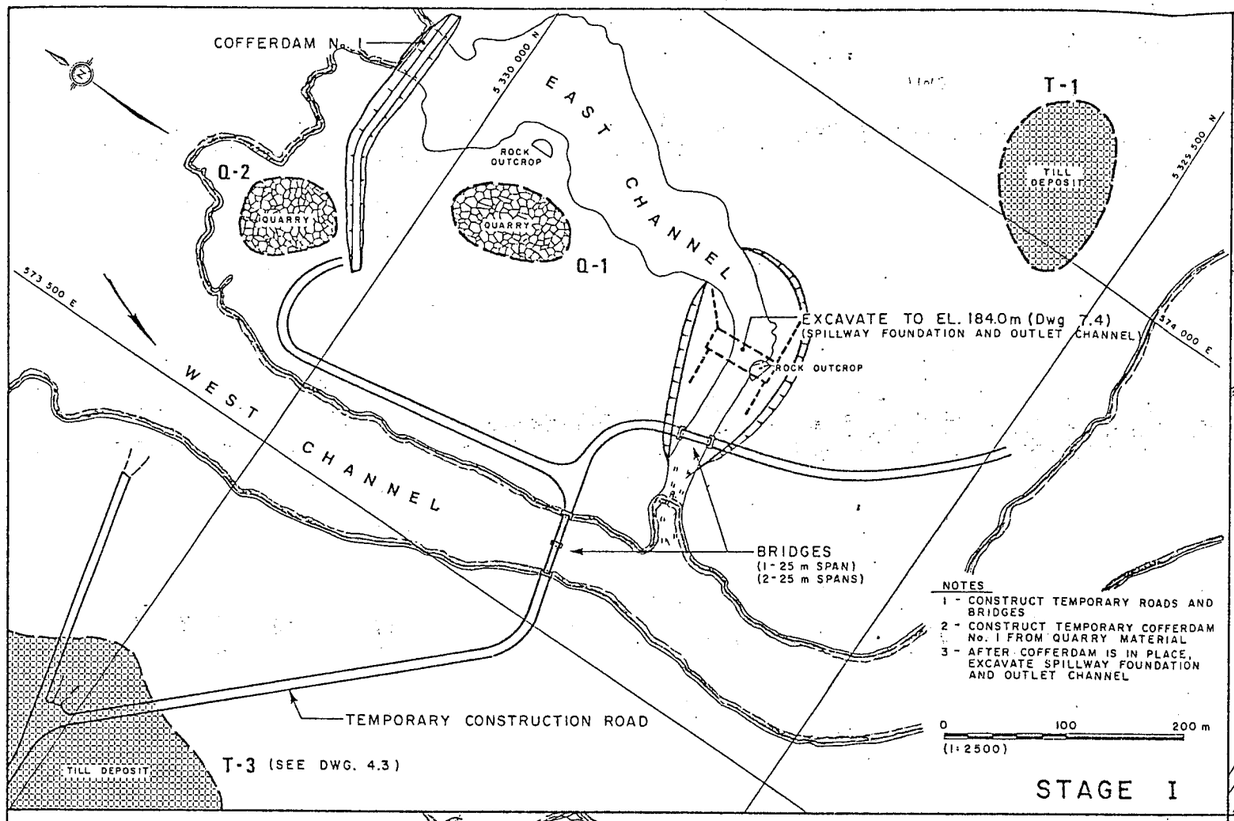
DRAWING 7.8
ROUND POND HYDROELECTRIC DEVELOPMENT
FEASIBILITY STUDY
- SINGLE LINE DIAGRAM -



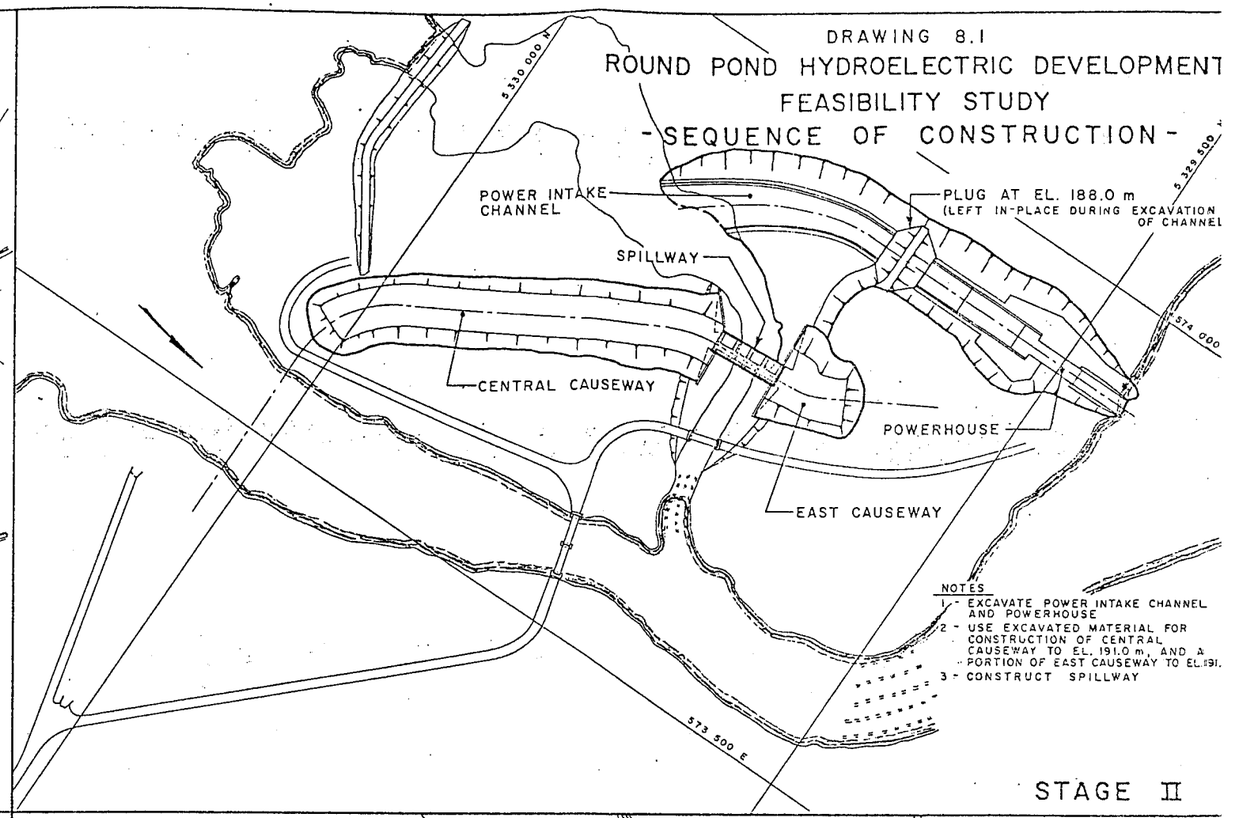
ISSUED: JUNE 88, SEPT. 88

DRAWING 8.1

ROUND POND HYDROELECTRIC DEVELOPMENT
FEASIBILITY STUDY
- SEQUENCE OF CONSTRUCTION -



STAGE I



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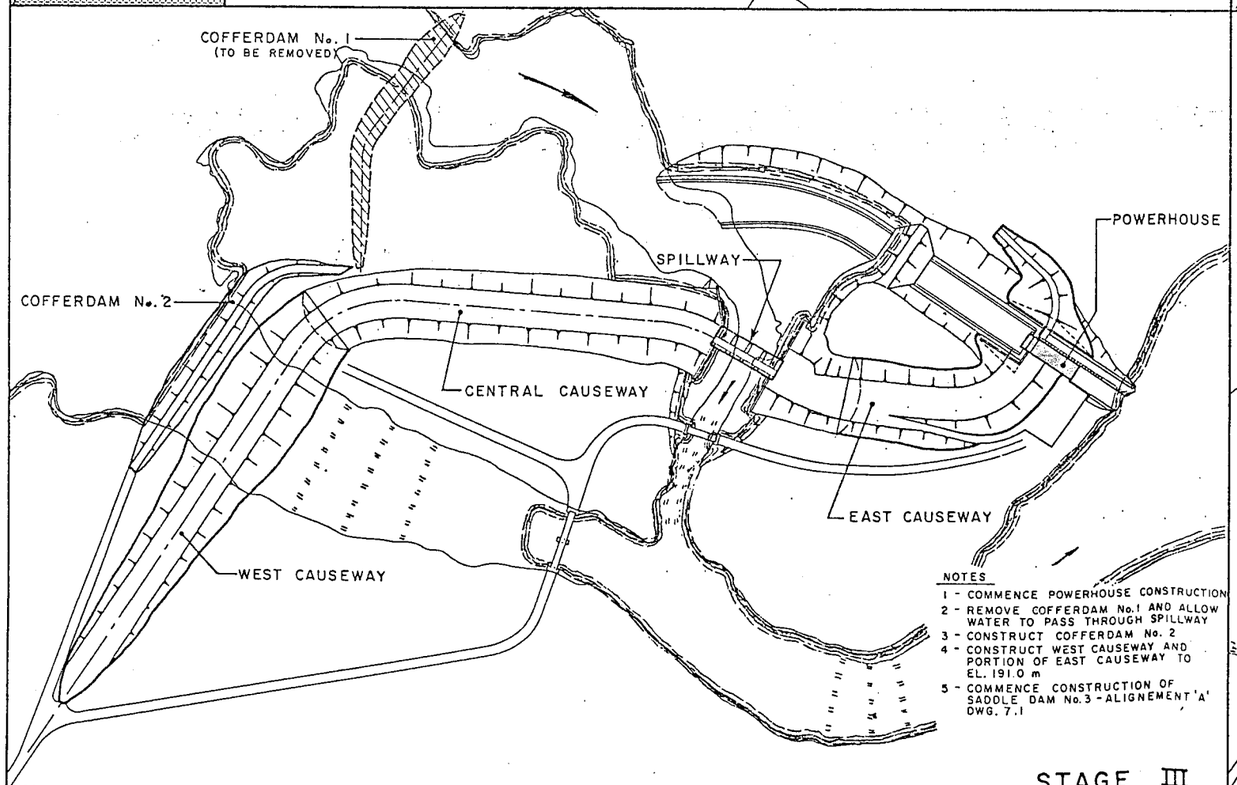

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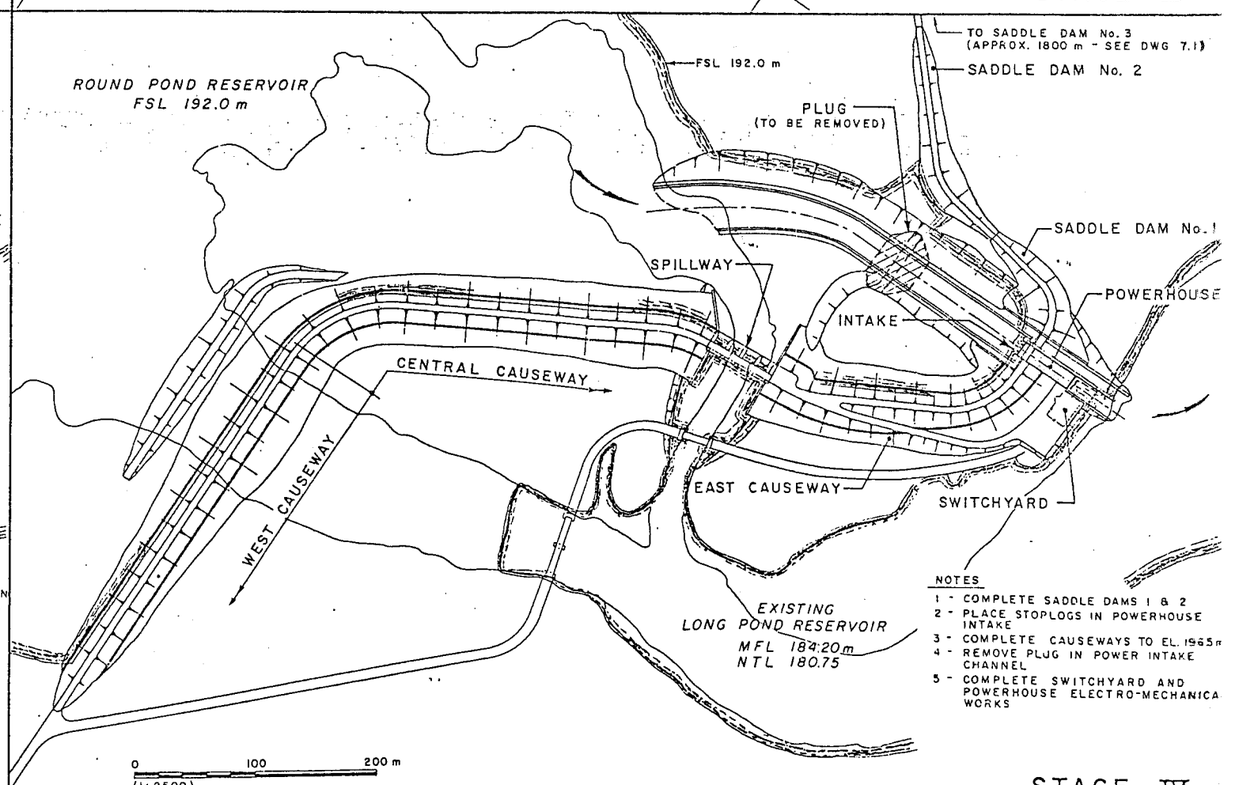

STAGE IV

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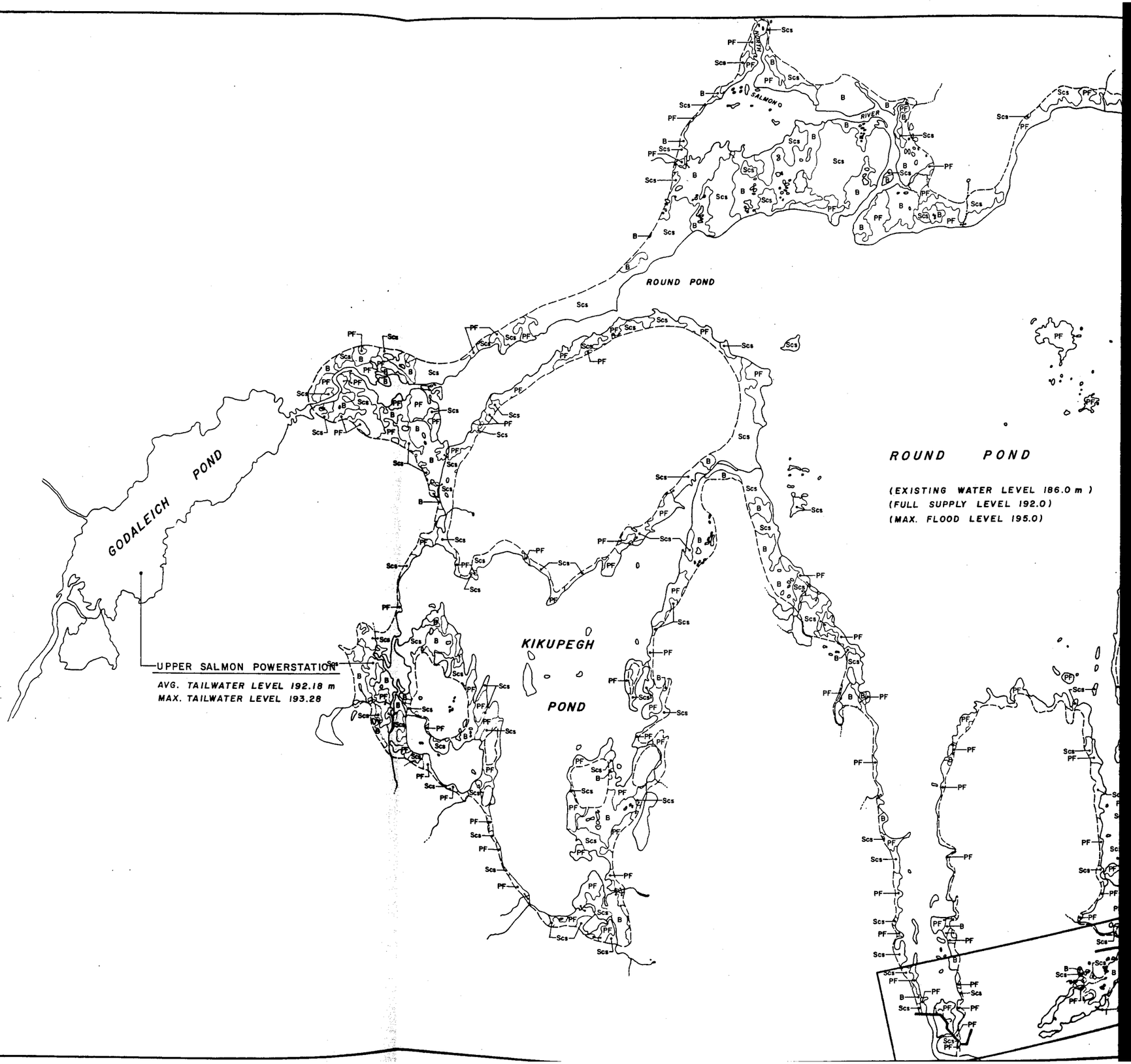

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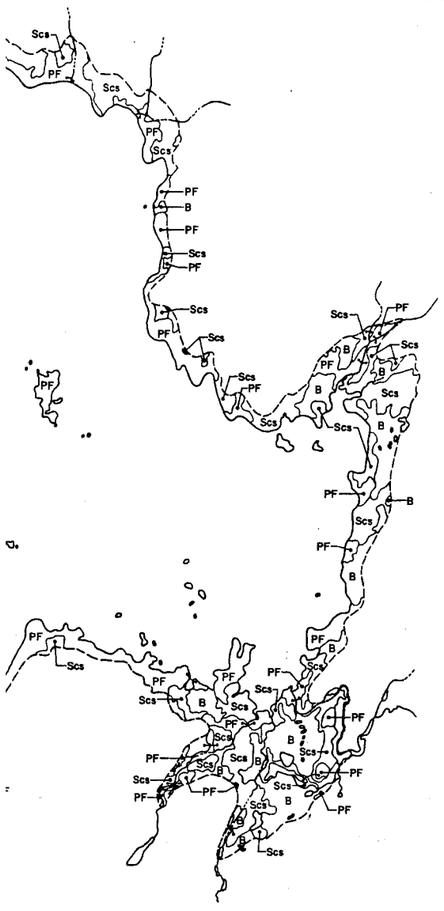
DRAWING 9.1

ROUND POND HYDROELECTRIC DEVELOPMENT
FEASIBILITY STUDY
- FORESTED AREA -

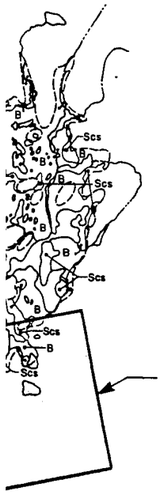


LEGEND

PF	PRODUCTIVE FOREST
Scs	SOFTWOOD SCRUB
B	ORGANICS (BOGS AND FENS)
-------	CLEARING LIMIT - horizontal distance of 1.0 m from FSL for steep topography.
	- 1.0 m from FSL along the bank slope for flat topography

NOTE :

THE FORESTRY INFORMATION SUPPLIED BY J.P. BOUZANE ASSOCIATES .



PROJECT AREA

SCALE 1:25 000

0 1 2 km

APPENDIX A

FEASIBILITY STUDY ALTERNATIVES

ROUND POND DEVELOPMENT

FEASIBILITY STUDY

APPENDIX A - FEASIBILITY STUDY ALTERNATIVES

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APPENDIX A

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A. FEASIBILITY STUDY ALTERNATIVES

A.1 INTRODUCTION

Appendix A describes the alternatives investigated during the feasibility study. During the course of the study a detailed review was made of the following:

1) Spillway arrangements
2) Number and type of units
3) Diversion and unwatering schemes
4) Saddle dam alignments
5) Structure alignments

The alternative access road routes were studied separately by Newfoundland and Labrador Hydro.

A.2 SPILLWAY ARRANGEMENTS

A.2.1 General

It was concluded from the prefeasibililty study that a conventional spillway arrangement should be adopted for the Development, because of the effect a dam break at Round Pond would have on the downstream structures around the Long Pond Reservoir of the Bay d'Espoir Development.

Three alternatives were therefore investigated for the spillway, namely:

Alternative A - a combined concrete overflow and gated structure;

Alternative B - a gated structure with nine gates 7.5 m high by 7.0 m wide.

Alternative C - a gated structure with four gates 7.5 high by 12.0 m wide.

Alternative D - a gated structure with seven gates 7.5 m high by 12.0 m wide with a reduced dam height.

A.2.2 Alternatives Considered

Alternative A - Combined Overflow and Gated Structures

This alternative comprises a concrete overflow structure constructed in the west channel and extending along the island for approximately 100 m. The total length of overflow structure is 200 m and would be capable of handling 50% of the PMF with a surcharge of 2.5 m over the crest. The balance of the design flood would be handled by a low ogee spillway containing four (4 No.) gates 7.5 m high by 7.0 m wide located in the east channel.

Alternative B - Gated Structure (7 No. gates)

With this alternative the flood would be handled by a low ogee gated structure with a surcharge of 3.0 m above the full supply level. The gates are the vertical lift type with a fixed hoist structure, and are 7.5 m high by 7.0 m wide spanning between 2.5 m wide piers. Nine gates were required to handle the PMF, resulting in an overall length of 90m for the structure. The structure would be located in either of the two channels.

Alternative C - Gated Structure (4 No. gates)

This alternative is similar to that described above for Alternative B except that four gates 7.5 m high by 11.0 m wide are used and the pier size would be increased to 3 m, giving an overall length of structure of approximately 60 m.

Alternative D - Gated Structure with a Reduced Dam Height

This alternative assumes that the design flood is discharged with no surcharge above FSL. Seven (7 No.) gates 7.5 m high by 12.0 m wide are required. The dam height is reduced to El. 193.5 m from El. 196.5m. This allows for a freeboard of 1.5 m above FSL.

A.2.3 Selected Arrangement

Comparative costs for each of the above alternatives are summarized in Table A-1. The gated spillway with four (4 No.) 7.5 m high by 11.0 m wide gates was found to be the least expensive and was therefore adopted for the selected layout.

A.3 UNIT SELECTION

A.3.1 Unit Selection

A.3.1.1 General

The gross head available for generation at the Round Pond site is in the order of 11.3 m and the flow established from optimization of the power facilities as described in Section 6.0 is 193 m3/s. The types of generating equipment suitable for these parameters are:

- S-type or tube turbines
- Straflo or Bulb (Pit) turbines
- Fixed blade propellor or adjustable blade propellor (Kaplan) units.

Fixed blade units were eliminated early on in the studies because of their lack of flexibility and hence lower overall efficiencies when used in single unit arrangements for run-of-river plants. The head at Round Pond is well below the minimum recommended for Francis units and consequently they were not considered in the study.

Both single unit and multiple unit arrangements were studied for S-type, Pit and Kaplan units and comparative costs for arrangements featuring each type of unit are summarized in Table A-2.

A brief commentary on each unit type and arrangements studied, follows.

A.3.2 Alternatives Considered

A.3.2.1 S-Type Units

For a plant flow of 193 m^3/s a minimum of three (3 No.) S-type units are required. Two units would be outside the upper limit of the flow range, which is approximately 70 m^3/s for a single unit.

The peak efficiency of an S-turbine is in the range of 91-92% and the efficiency curve is flat since it is possible to have regulation from both the wicket gates and the runner. A typical efficiency curve is shown in Figure B.1.

A typical arrangement for three (3 No.) S-turbines is given on Dwg. A-1. This arrangement costs $5.75 million more than the single Pit alternative and $4.13 million more than the double Pit arrangement. As illustrated in Table B.2 the main difference in cost is attributable to the equipment.

A.3.2.2 Straflo-Bulb (Pit) Turbines

Bulb units are generating sets through which water flows axially from the intake to the draft tube outlet. The equipment itself comprises a horizontal shaft Kaplan turbine with either fixed or movable guide vane rings, driving an A.C. generator mounted inside a waterproof housing placed in the centre of the water passage (hence the term "Bulb"). The Pit turbine follows essentially the same principle but instead of the generator being completely

enclosed in the "bulb" it is housed in a steel "pit" which is accessible from the powerhouse floor. The development of the "Pit" turbine in the past few years from the "Bulb" concept has been due to the increased reliability of planetary gearing systems, which has enabled the physical size of the generator to be reduced.

The efficiency curve for the Pit turbine is flat like the S-type (ref. Fig. A.1) but has a higher peak efficiency (in the order of 93-94%). A double unit arrangement has an increased overall efficiency when operating in the flow range from 50 m^3/s to 115 m^3/s. The corresponding gain in energy production, however, would not be significant since this range of flows lies in the 85-100% exceedance range of the flow duration curve.

The double unit alternative was found to cost $1.5 million dollars more than the single unit. The cost difference was attributable entirely to the equipment cost.

The Straflo turbine is similar in arrangement to the Bulb turbine but features a generator housed in a rim attached to the propellor. It is usually fitted with non-adjustable blades, since to enable adjustments to be made to the blade settings, bushings are required at connections between the blade and rim, which can pose problems with maintenance.

Standardized designs for this unit are available only up to design flows of approximately $150m^3/s$ and a minimum of two units would therefore be required at Round Pond. It was therefore concluded at an early stage in the study that the Straflo unit would have no cost or operational advantages over the Pit turbine and was not studied in detail.

A.3.2.3 Kaplan Units

A typical arrangement for a Kaplan unit is given on Dwg No. A.2. As illustrated in Table B-2 the single Kaplan costs $4.7 million more than the single Pit alternative. The cost difference is divided approximately equally between the civil and equipment costs. To check this a comparison was made between Kaplan and Pit turbines using ECOP, a computer program developed for the James Bay Energy Corporation to compare installations with Francis, Kaplan and Pit Turbines. This also indicated that the arrangement with the single Pit turbine is the most economic, giving a cost advantage of $3.5 million.

The efficiency of the Pit turbine is slightly higher than the Kaplan over the full range of flows as indicated on Figure A.1. However the efficiency advantage gained by using the Pit unit is offset to an extent by the larger exit velocity from the draft tube resulting in a higher exit loss.

A.3.2.4 Selected Alternative

This study confirmed the findings of the prefeasibility study that the single Pit unit is the least costly alternative. The cost difference between the single unit and double unit Pit arrangements is $1.5 million.

There are advantages, however with a double unit arrangement in that lost production during maintenance is minimized and also there is an overall gain in efficiency at lower flows. The monetary value of this loss in energy was assessed as follows:

	Energy Loss (GWh)	Loss in Revenue ($) ($0.083/KWh)
Net loss in energy for the Upper 80-100% range of the flow duration curve	0.175	14,525
Loss in energy for maintenance assuming downtime of 1 week (using avg. October flow)	1.500	124,500
TOTAL	1.68	139,025

Compared with annual charges of $165,000/annum on a capital investment of $1.5 million at 10% discount rate of 25 years, the single unit still has a marginal cost advantage.

The single pit turbine was therefore adopted for the layout.

TABLE A.1

SPILLWAY ALTERNATIVES - COMPARATIVE COSTS

ALTERNATIVE	DESCRIPTION	COMPARATIVE COST
A	Concrete overflow and gated spillway	9,700,000
B	Gated spillway (9 gates - 7.5m high x 7.0 m wide)	7,200,000
C	Gated spillway (4 gates - 7.5 m high x 12.0 m wide)	4,800,000
D	Gates spillway (7 gates - 7.5 m high x 12.0 m wide with reduced dam height)	5,800,000

TABLE A.2

UNIT SELECTION - COMPARISON TABLE

ITEM	SINGLE PIT	DOUBLE PIT	S-TYPE	KAPLAN UNITS
* Civil Costs	4,288,750	4,141,100	4,878,300	6,500,000
** Equipment Costs	8,900,000	10,520,000	14,646,000	11,125,000
TOTAL	13,188,750	14,661,100	19,524,300	17,873,200

* Civil costs include overburden and rock excavation in powerhouse and tailrace and concrete, formwork and rebar costs.

** Equipment costs include "water to wire" packages for the turbine and generator for the Pit and S-Type units. This includes steel pit structures, horizontal synchronous generators c/w excitation turbine/governor controls, low voltage switchgear and main transformer. The cost of the main transformer is not included in the equipment costs for the Kaplan unit.

A.4 DIVERSION AND UNWATERING SCHEMES

A.4.1 General

The two channels at the outlet of Round Pond offer several alternative schemes for diversion during construction.

Approximate cross sections of the channels were prepared from the limited survey carried out during the prefeasibility field program, from which the preliminary rating curves shown on Figure A.3 were developed. A rating curve was available for the government gauge close to the West Causeway axis but unfortunately was only of limited use as the value of the reference datum could not be determined.

The alternative schemes investigated are summarized as follows:

Alternative 1 - Use of the east channel alone for the first stage of diversion to allow construction of the spillway structure in the west channel behind a single cofferdam.

Alternative 2 - Use of the east channel for the first stage of diversion with some improvements at the control section to increase the flow capacity to allow construction of the spillway structure in the west channel as in Alternative 1 above.

Alternative 3 - Construction of the spillway in the west channel in two stages.

Alternative 4 - Construction of the spillway in the east channel while diverting the flow through the west channel.

The schemes are described in detail in the following section.

A.4.2 Alternatives Considered

Alternative 1

This alternative considers the use of the east channel only, for the first stage of diversion. The west channel is closed completely to allow construction of the spillway piers, abutments and spillway bridge and the portion of the Causeway in the river channel and on the channel banks up to El. 187.0. The embedded parts for the spillway stoplogs would be installed at this stage. After completion of the permanent works, the cofferdam would then be removed and the flow passed through the low ogee spillway. A second cofferdam would then be constructed in the east channel, to allow construction of the causeway to proceed. Installation of the spillway gates and embedded parts would take place behind the stoplogs, later in the construction period.

In order to minimize the ingress of water into the quarry and the causeway foundation, the upstream water level would be restricted to El. 187.0. From the rating curve for the east channel this corresponds to a flow of 300 m^3/s which is approximately equivalent to a 2-year return period.

There are obvious disadvantages with this scheme. With protection only up to a 2-year return period, there is a risk of overtopping the cofferdam even if a single construction season to complete the permanent works in the west channel is assumed. The cofferdam would therefore require armouring on the downstream side in a manner similar to that indicated for the rockfill overflow structure indicated on Drawing B-1.

Another disadvantage is possible damage to the temporary construction bridge crossing the east channel on the downstream side of the spillway. The narrow section upstream of the bridge could cause a considerable rise in water level which could wash out the abutments.

A.4.2.2 Alternative 2

This is essentially the same scheme as Alternative 1, but improvements would be carried out on the east channel to increase the flow capacity to 500 m3/s (1 in 20 year flood). The improvements would comprise widening the control section at the start of the rapids, by blasting the right side of the V-shaped channel bottom to a width of 40 m.

Because the flow is confined to the east channel there remains the risk of flooding the construction road and bridge and the area upstream below El. 187.0 m.

A.4.2.3 Alternative 3

This scheme features construction of the spillway in two stages in the west channel and is similar to that described in Appendix B - Prefeasibility Study. A fill cofferdam extending from the right bank would be constructed so as to enclose sufficient working area to enable construction of two bays of the spillway. The flow would meanwhile pass through the remaining part of the west channel and the whole of the east channel. The upstream water level would again be restricted to 187.0 for the reasons previously discussed. The flow passing through the open part of the west channel would be approximately 200 m3/s. The downstream slope on the section of cofferdam parallel to the flow would be armoured with heavy rip rap to stop erosion of the impervious material.

On completion of the two spillway bays the cofferdam would be removed and the remaining part of the west channel closed with a second cofferdam. In order to keep the two completed bays from being obstructed by fill from the cofferdam, either of the following methods would be used to close the cofferdam against the completed spillway piers:

(1) a timber crib wall extending to the fill part of the dam. (as indicated on Drawing B-5)

(ii) a cellular type cofferdam extending to the fill.

When the spillway and the causeway section are completed the cofferdam is removed. The causeway in the East Channel is then constructed behind a second cofferdam with the flow passing through the completed spillway.

With the two stage cofferdam the spillway would take longer to construct because of the restricted working area, removal of the first stage cofferdam and the more complicated sealing arrangement against the piers of the completed spillway structure.

A.4.2.4 Alternative 4

This scheme is illustrated stage by stage in Dwg. 8.1 in the main report.

With this scheme the spillway is constructed in the east channel. A cofferdam is constructed over the northern tip of the island to provide protection to the quarry, the causeway foundation and the temporary access road and bridge downstream.

Because the west channel is considerably wider than the east channel the flow capacity is correspondingly greater. For an upstream water level of 186.5 m the west channel has a capacity of 500 m^3/s (1 in 20 year flood).

The risk of washing out the construction road and bridge downstream, is much less with this scheme because of the wider channel.

After completion of the spillway in the east channel, the cofferdam would be removed allowing the flow to pass through the spillway. The causeway in the west channel would then be completed behind a second cofferdam.

A.4.2.5 Selected Scheme

Alternative 4 is considered to offer the best security for the works during construction. There is less risk to the construction road downstream and the area immediately downstream of the cofferdam is safe from inundation.

A.5 SADDLE DAMS

A.5.1 Alternatives Considered

In order to effect closure at El. 192.0 m three saddle dams are required; two minor freeboard structures approximately 2.0 m high close to the powerhouse and a major structure spanning a low lying area some 1.5 kilometers to the east of the project area.

Two axes, as identified on Drawing B-4 were investigated for the main saddle dam. These are briefly described as follows:

Alignment A

This axis is some 1050 m long and spans a large area of fen and bog just south of the easterly tip of Round Pond. The geotechnical investigation revealed depths of peat up to 2.8 m overlying a layer of cobbles and boulders. Rock was encountered at a depth of about 4.0 m at the deepest section of bog and the ground water level along the lowest part of the alignment was high.

Alignment B

Alignment B is located some 1 kilometer south of Alignment A and is approximately 800 m long. The quantity of bog along this alignment is considerably less with the maximum depth recorded in the order of 1.0 m. The groundwater table also appears much lower than at the upper alignment.

A.5.2 Selected Alternatives

The foundation excavation and embankment fill quantities are considerably less at Alignment A, resulting in an overall cost saving of approximately $300,000. There was some concern before the drilling program that this cost advantage could be offset by higher dewatering cost, because of the deep section of bog at the upper alignment. However this concern has now been allayed since the field investigation. The borehole information indicates that the rock level is 1 m above the level of Round Pond and therefore it does not appear that well points will be necessary for dewatering. Alignment A is therefore selected for the project layout.

A.6 STRUCTURE ALIGNMENTS

A.6.1 Causeways

The alternative causeway alignments considered during the prefeasibility study were again reviewed at the feasibility phase. Since the concrete overflow dams had been rejected as being too costly the lower alignments were now subject to less penalty from increased concrete quantities. Consequently it was found that with the gated spillway located in the east channel, Alignment 2, based on the fill quantities given in Table B-2, now had a cost advantage of $280,000 over Alignment 1. This cost difference was not sufficient however, to warrant changing the alignment, since the bedrock elevation was believed to be higher at Alignment 1, giving a more positive cut-off for the causeway. This bedrock level was later confirmed by the drilling program. The results of the material testing carried out during the field investigation also revealed granular layers in the overburden material which also confirmed the choice of Alignment.

Alignment 3, although having much less fill volume, had a larger concrete quantity for the spillway and a higher cost associated with the diversion because of the need to excavate a diversion channel. Prior to the feasibility study field investigation it was

found that the cost saving in embankment fill was offset by these additional costs.

A.6.2 Structure Layout

The structure layout prepared prior to the drilling program is shown on Dwg A-4. In preparing this layout it was assumed that the overburden in the intake channel was 3-5 m deep based on data collected during the preliminary field investigation. To utilize the rock from the intake channel excavations a zoned rockfill dam was assumed for the causeway embankments, as illustrated on Dwg. A-3. It was subsequently found from the results of the drilling program that the overburden depth was in fact in the order of 9 m and this necessitated the following changes to the layout:

1) Since there was now insufficient rock available from excavation to use in the construction of a rockfill zoned embankment, the design was changed to a homogeneous dam, constructed from glacial till obtained from borrow areas adjacent to the site. (See Dwg. 7-3).

2) The alignment of the dam on the left side of the east channel had to be revised to accommodate the flatter excavation side slopes in the intake channel.

3) Higher retaining walls were required at the powerhouse to retain the dam embankment and backfill near the powerhouse structure. This has resulted in an increase in concrete quantities of approximately 8,000 m^3.

There could be a cost saving of around $2,000,000 if the spillway and powerhouse were located in the same channel in conjunction with structure Alignment 3, thus eliminating two retaining walls. However, this alternative would require further geotechnical investigation to determine the bedrock elevation at the new spillway location.

The final project layout accepted for the feasibility study is shown on Dwg. 7.1 and Dwg. 7.2.

FIGURE A-1

COMPARATIVE EFFICIENCY CURVES

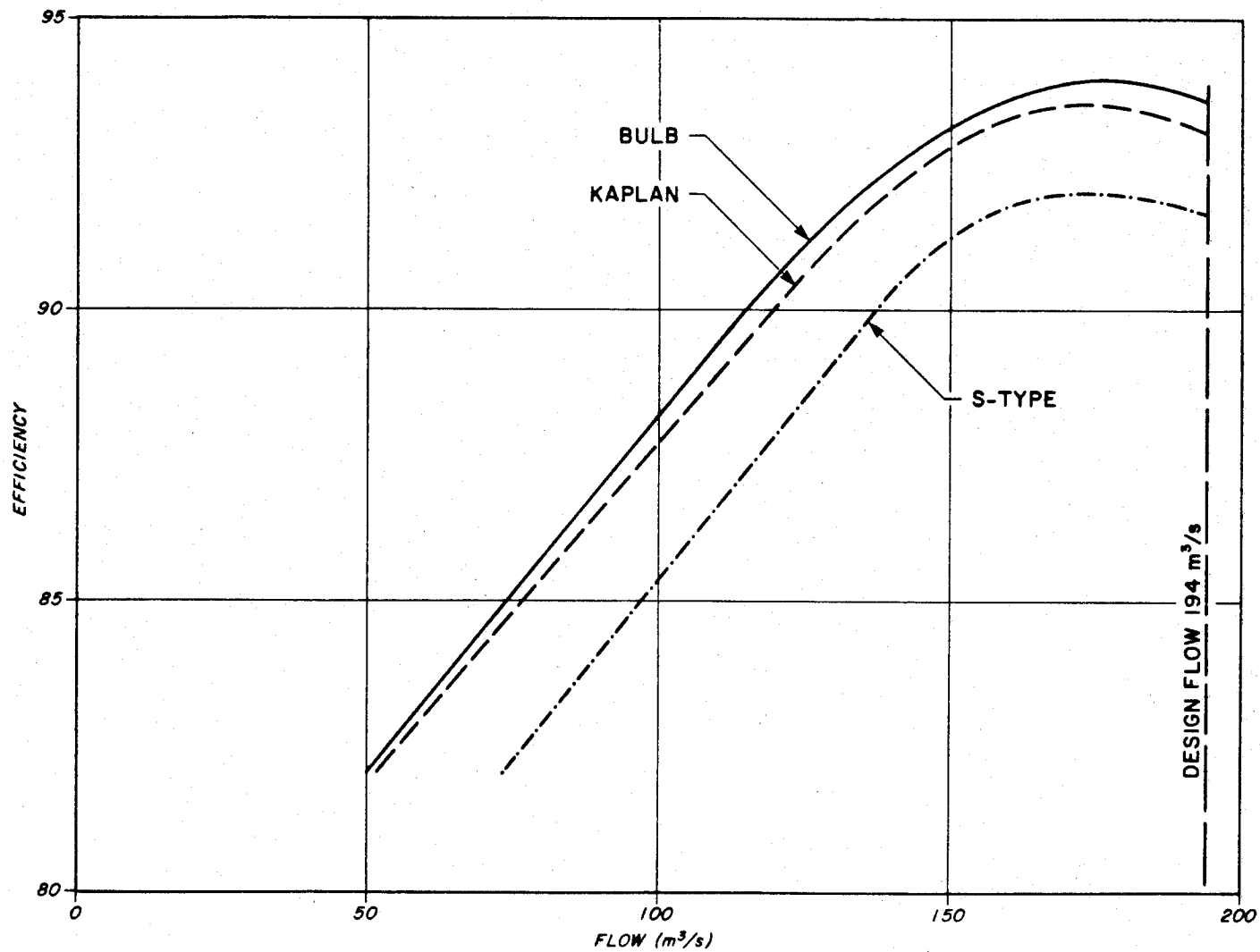


FIGURE A - 2

EFFICIENCY CURVE FOR DOUBLE BULB UNIT

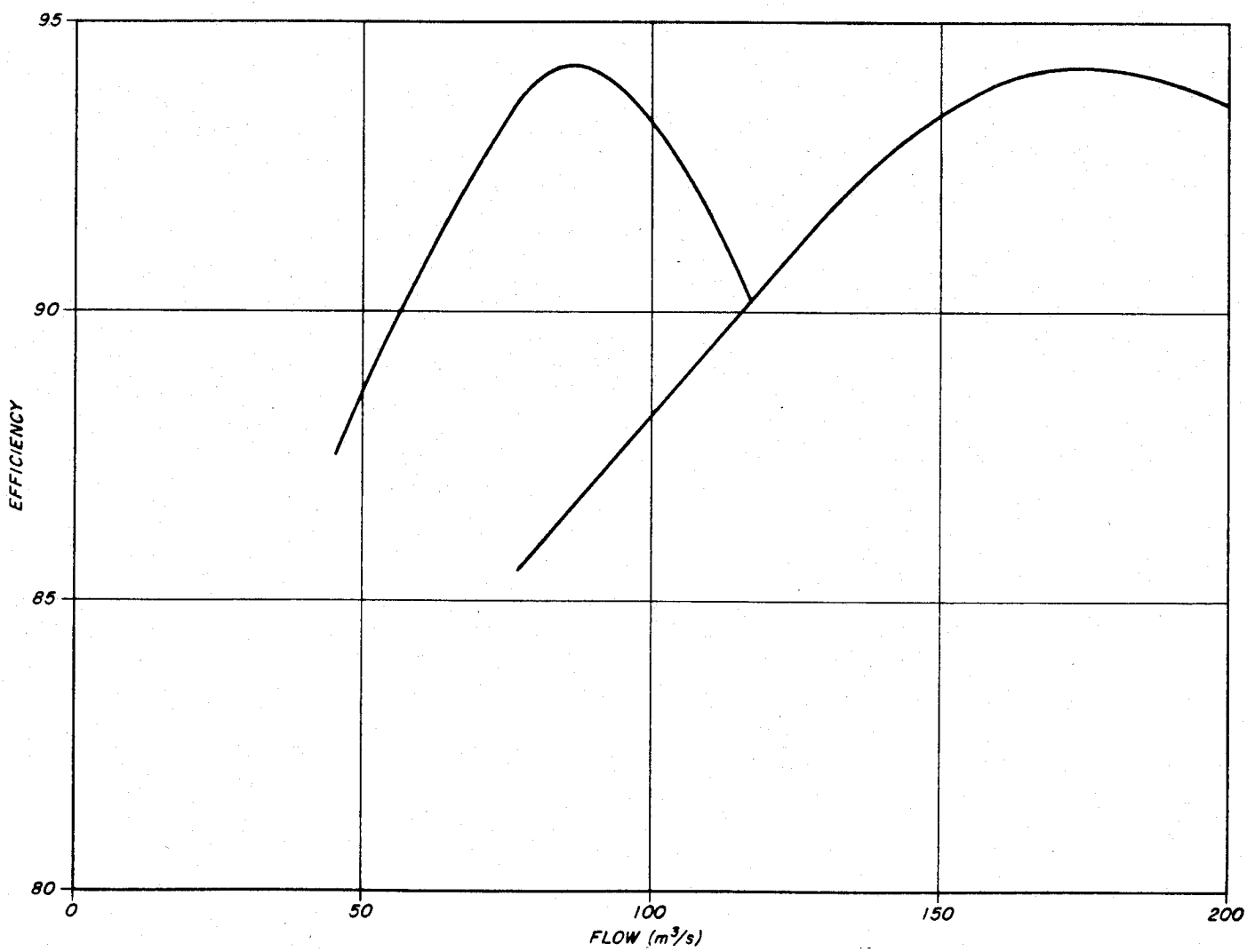
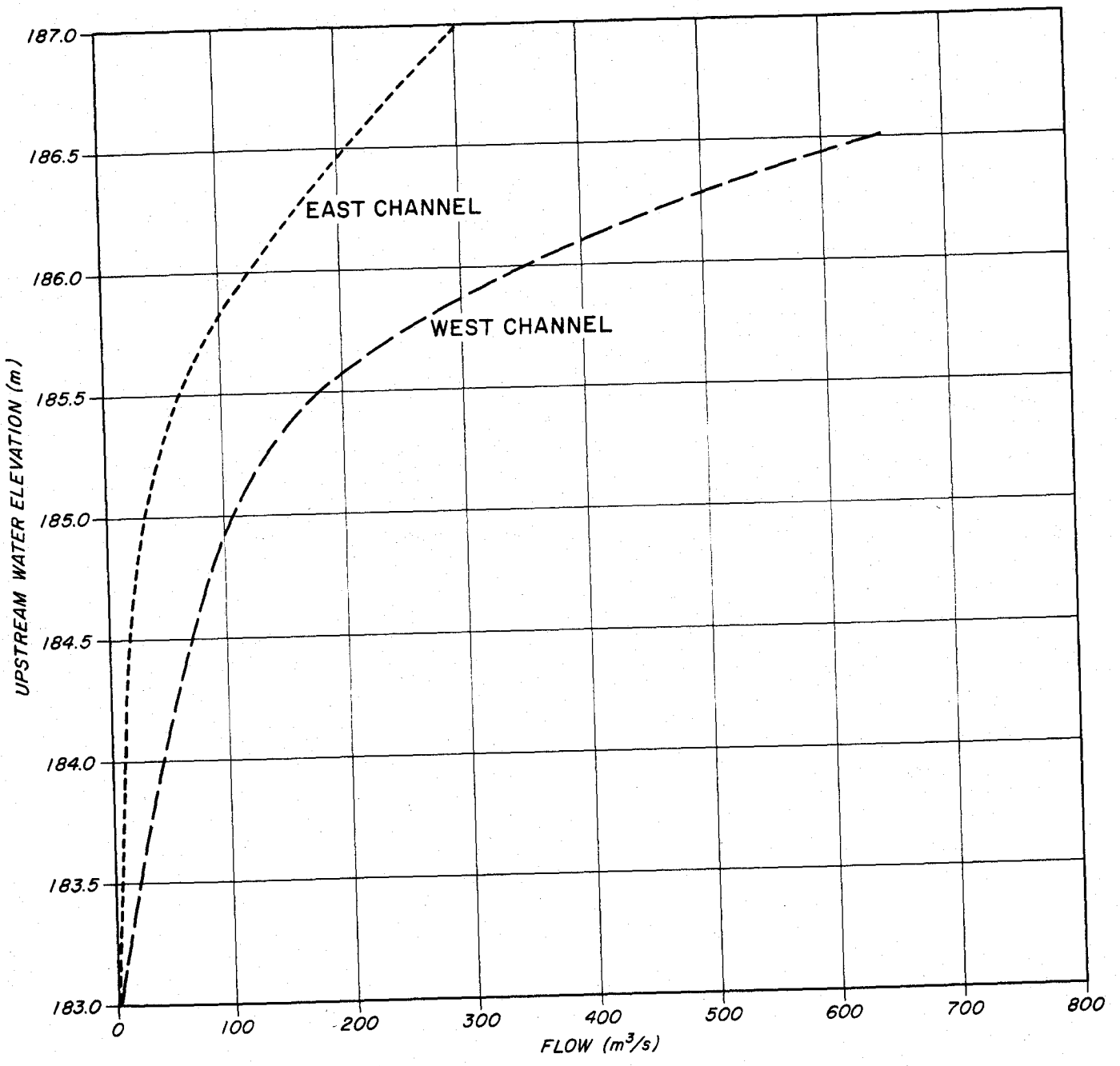


FIGURE A-3

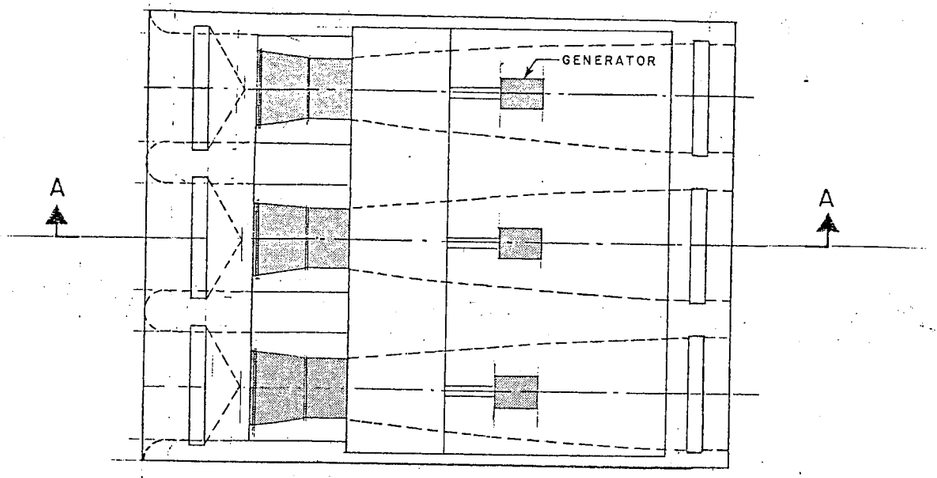
RATING CURVE FOR EAST AND WEST CHANNELS



DRAWING A-1

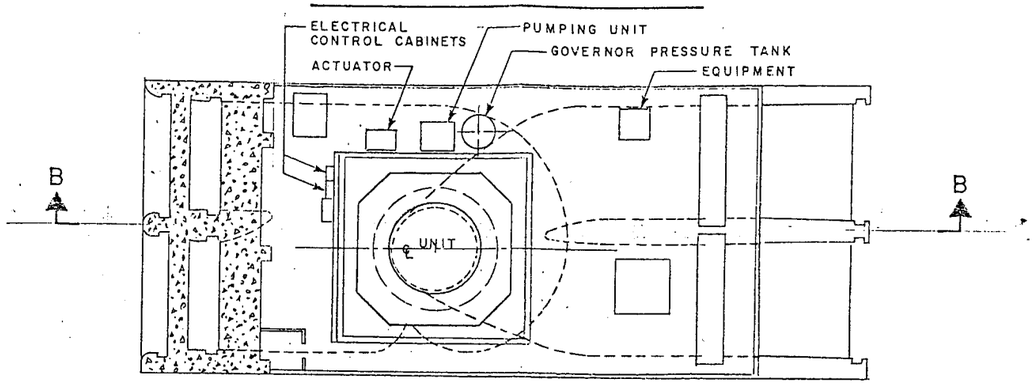
ROUND POND HYDROELECTRIC DEVELOPMENT
FEASIBILITY STUDY
- POWERHOUSE ALTERNATIVES -

S - T Y P E

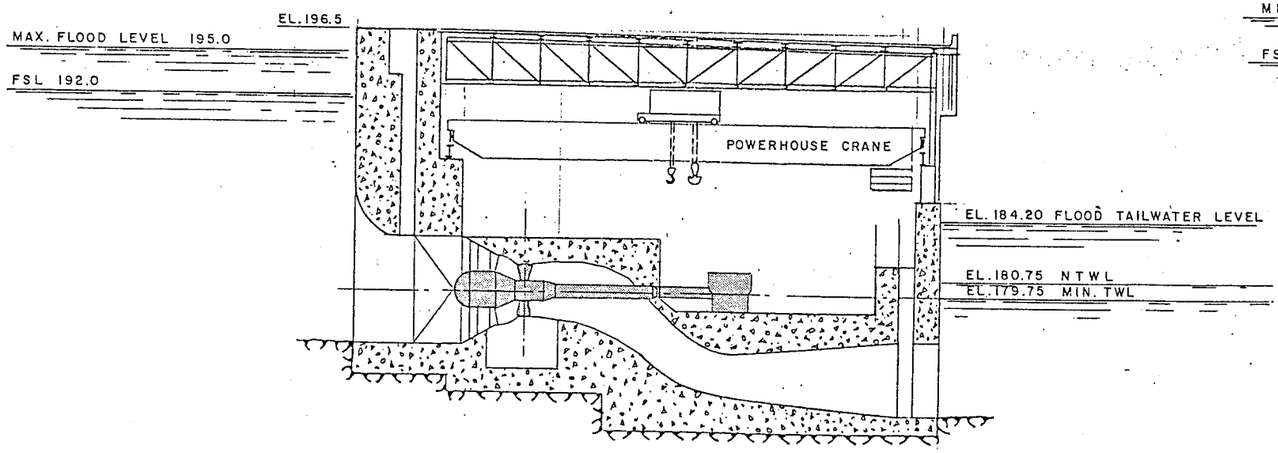


PLAN

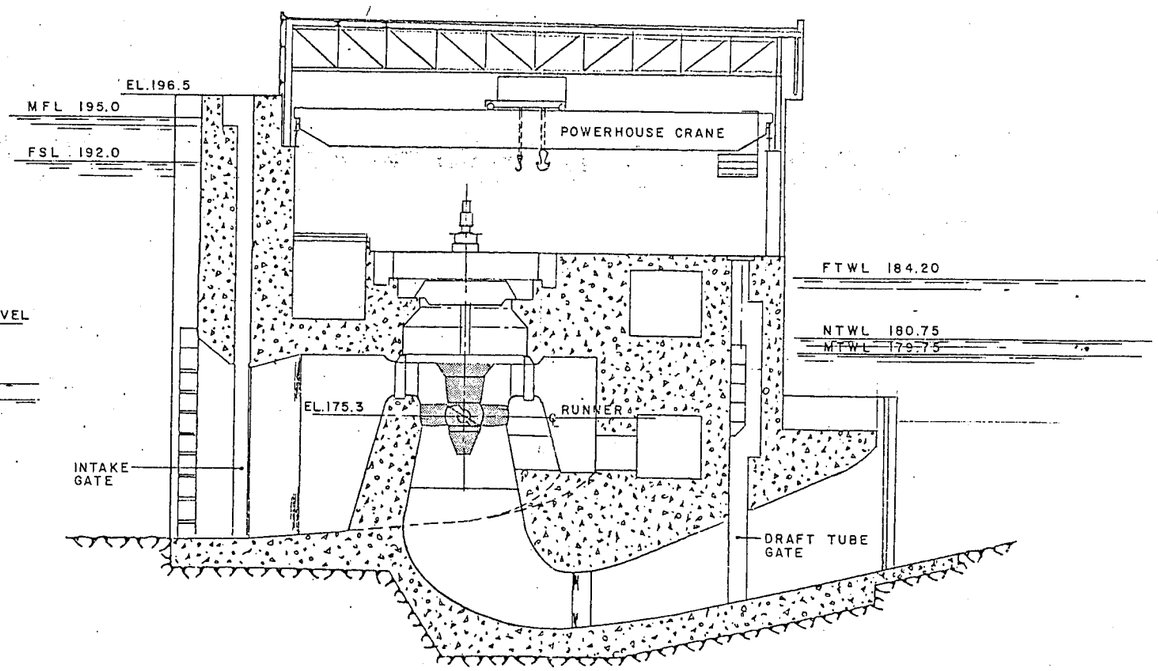
K A P L A N



PLAN



SECTION 'A-A'



SECTION 'B-B'

ISSUED: JUNE 88, SEPT. 88

0 2 4 6 8 10 20 (1:200)

DRAWING A-2

ROUND POND HYDROELECTRIC DEVELOPMENT

FEASIBILITY STUDY

STRUCTURE LAYOUT

D (STRUCTURE LAYOUT PRIOR TO DRILLING PROGRAM)

ROUND POND RESERVOIR
FSL 192.0 m

COFFERDAM No. 1

COFFERDAM No. 2

C

CENTRAL CAUSEWAY

WEST CAUSEWAY

C (DWG A-3)

A

TO UPPER SALMON ROAD 12 km±

TEMPORARY CONSTRUCTION ROAD

573 300 E

5 330 000 N

574 000 E

TO SADDLE DAM No. 3 1.5 km±

A (DWG A-3)

D (DWG A-3)

FSL 192.0 m

INTAKE CANAL

I.E. 184.0

I.E. 179.5

4.2 %

SPILLWAY

EXCAVATE TO EL. 183.0

SLOPE ROAD 6.5% GR.

EAST CAUSEWAY

BRIDGE
(1-25 m SPAN)

BRIDGE
(2-25 m SPANS)

ROCK FILLED TIMBER CRIBS

SADDLE DAM No. 2

SADDLE DAM No. 1

RETAINING WALL

INTAKE

FISH SCREEN

FISH PASSAGE

POWERHOUSE

FISH SCREEN

TAILRACE

EL. 185.5 m

SWITCHYARD

EL. 184.75

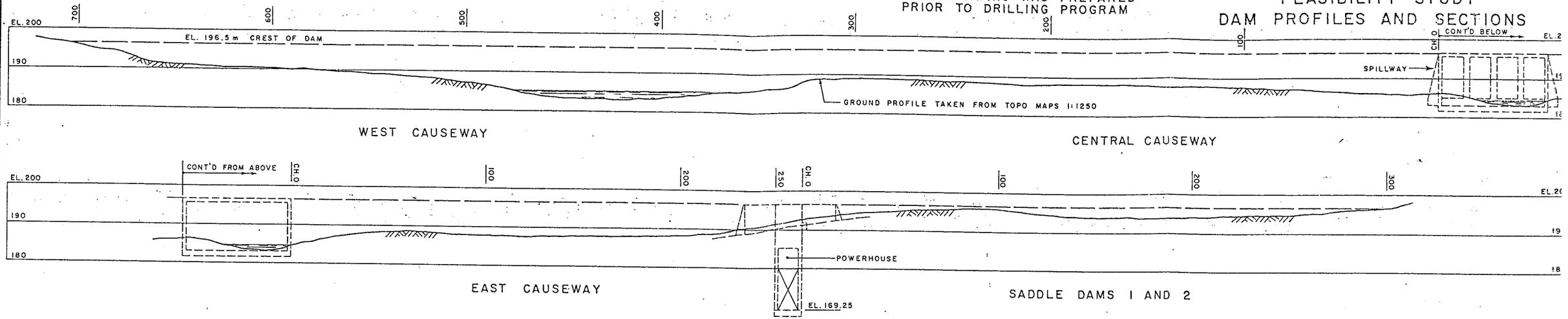
LONG POND RESERVOIR
EXISTING
MFL 184.20 m
NTL 180.75

SCALE 1:1250
0 50 100 m

DRAWING A-3

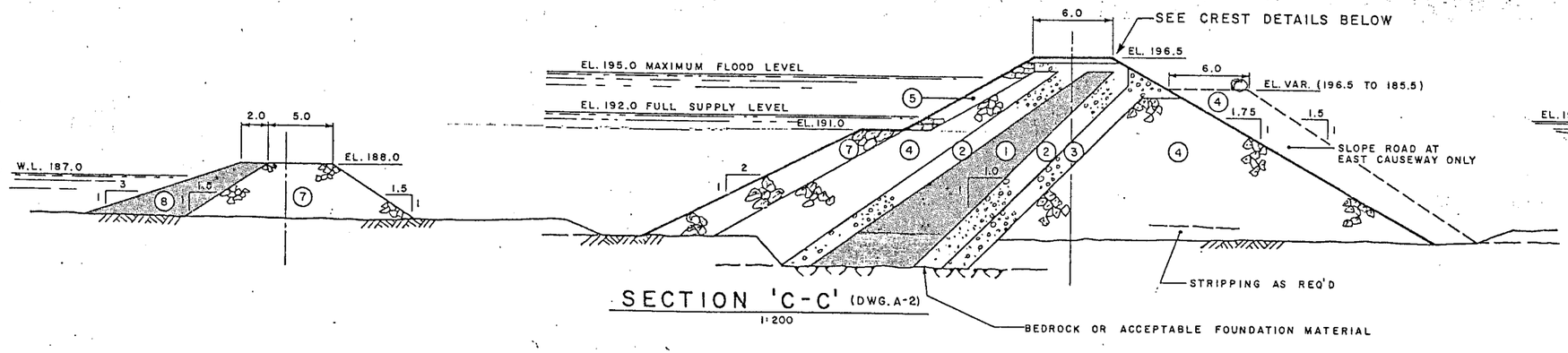
ROUND POND HYDROELECTRIC DEVELOPMENT
FEASIBILITY STUDY
DAM PROFILES AND SECTIONS

NOTE: THIS DRAWING WAS PREPARED PRIOR TO DRILLING PROGRAM



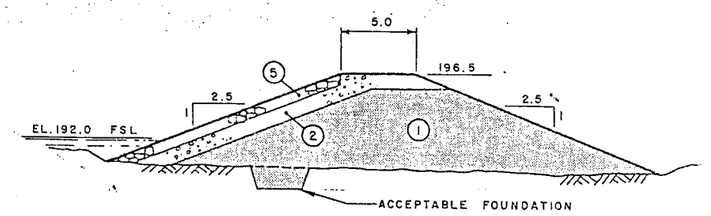
LONGITUDINAL PROFILE 'A-A' (DWG. A-2)

SCALE: HOR. 1:1000 VERT. 1:500



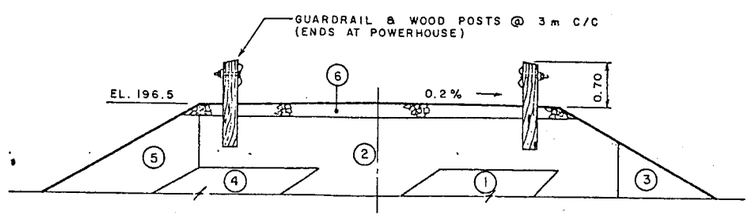
SECTION 'C-C' (DWG. A-2)

1:200



SECTION 'D-D' (DWG. A-2)

1:200



CREST DETAILS

1:50

LEGEND

1. ROLLED IMPERVIOUS FILL (TILL)
2. TRANSITION FINE (SEMI PERVIOUS)
3. TRANSITION COARSE (PERVIOUS)
4. COMPACTED ROCKFILL
5. RIPRAP
6. GRANULAR SURFACING
7. DUMPED ROCKFILL
8. DUMPED IMPERVIOUS

0	2.5	5	(1:50)
0	10	20	(1:200)
0	25	50	(1:500)
0	50	100	(1:1000)

ISSUED: JUNE 88, SEPT. 88

APPENDIX B

PREFEASIBILITY STUDY

ROUND POND DEVELOPMENT

FEASIBILITY STUDY

APPENDIX B - PREFEASIBILITY STUDY

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TABLE B.2 Causeway Alignments Fill Quantities

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APPENDIX B

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Dwg. B.5	Diversion Scheme

B. PREFEASIBILITY STUDY

B.1 INTRODUCTION

The following engineering work was carried out during the prefeasibility study:

1) Review of spillway type
2) Review of dam type
3) Investigation of alternative structure alignments
4) Development of Layouts
5) Diversion and unwatering
6) Selection of unit type and number of units
7) Energy estimates
8) Optimization of dam height and turbine flow
9) Capital cost estimate and monthly cash flow.

The work was carried out to an extent necessary to provide a capital cost estimate and estimate of available energy to allow Newfoundland and Labrador Hydro to make a commitment to proceed with further work.

The results of the prefeasibility study were presented and discussed with HYDRO in St. John's, Newfoundland on October 5, 1987 in the form of sketches, results of the power and energy work and a detailed cost estimate. The completion of the prefeasibility study with a prefeasibility report was not requested by HYDRO.

A summary of the work carried out is given in the following sections.

B.1.1 Review of Spillway Type

The original concept of an earth fuse plug spillway to pass a portion of the design flood, as proposed by Acres in their preliminary study was abandoned, because of the potential for damage to the Bay d'Espoir structures downstream by release of the increased volume of water. This volume of water would now be retained by the higher structures at Round Pond due to the increased head.

It was considered, however, that a reinforced rockfill overflow structure would be acceptable, since, although some damage would be expected to occur in a major flood, this would not constitute a major breach. This spillway type was therefore examined in combination with concrete overflow structures to pass the design flood, and a cost comparison made with a low ogee gated structure. Both these alternatives are described in detail as follows:

Alternative A - Combination of Concrete and Rockfill Overflow Structures

This alternative features concrete overflow structures constructed in the east and west channels at the outlet to Round Pond and a rockfill overflow embankment 600m long constructed in the saddle dam area.

The concrete overflow structures are 220 m long and capable of passing 50% of the PMF (1650 m^3/sec) with a surcharge of 2.5 m over the crest. The balance of the PMF would overtop the rockfill structure in the saddle dam area. The downstream face of the dam is reinforced with steel mesh and anchored with steel bars driven into the rockfill; the toe of the dam is secured by anchors drilled into the rock foundation. Cross sections of the rockfill and overflow structures are shown on Dwg. B-1 and a layout featuring this spillway arrangement is presented in Dwg. B-2.

Alternative B - Gated Structure

With this alternative the flood would be handled by a low ogee gated structure with a surcharge of 3.0 m above the full supply level. The gates are the vertical lift type with a fixed hoist structure, 7.5 m high spanning 7.0 m between 3 m wide piers. Nine gates are required to handle the PMF, giving an overall length of 90 m for the structure. A layout featuring this spillway arrangement is presented in Dwg. B-3.

Both types of spillway were found to have approximately the same cost (as indicated in Table B-1 on Page B-10) and therefore the combined rockfill/concrete overflow structure was eliminated from further study.

B.1.2 Dams

a) Causeways

Both homogeneous and zoned construction were considered for the causeways. These are described as follows:

Homogeneous Dam

The embankment is constructed of compacted glacial-till with a rip rap face to protect the upstream side. A transition zone separates the rip rap from the till. Slopes of 2.5 m horizontally to one vertically have been adopted on both the upstream and downstream side. A cut off trench is excavated to a suitable till foundation and filled with compacted till. A typical cross section is shown on Dwg. 7.3.

b) Zoned Dam

This comprises an outer rockfill shell enclosing an impervious core extending to bedrock. Transition zones separate the

rockfill from the impervious core. In order to keep the crest width to a minimum the core is inclined towards the upstream face and consequently to counteract potential destabilizing forces, a dumped rockfill bund is placed against the upstream toe. A typical cross section of the zoned embankment is given on Dwg. A-3.

B.1.3 Causeway Alignments

The alternative alignments considered for the prefeasibility study are presented in Dwg. B-4 and are described briefly as follows:

Alignment 1

This alignment is basically the same as the one presented in the Acres report. It follows the high ground on the island and on the right bank of the east channel.

Alignment 2

This alignment is approximately 200 m downstream of Alignment 1 in the west channel.

Alignment 3

This alignment is 30 m downstream of the confluence of the east and west channels.

Fill volumes for the embankments for the above alignments are summarized in Table B-2, Page B-11.

Alignment 1 although having the largest volume of fill is some 2 m higher than Alignment 3 and 1.5 m higher than Alignment 2 at the west channel. Therefore the additional fill quantity for this alignment is more than offset by the saving in

concrete quantity for spillway structures constructed in the west channel. In addition test pits excavated during the field investigation indicated that rock was close to the surface across a large section of the island. Therefore, since at this stage overflow concrete dams were still being considered as viable alternatives to pass the design flood, Alignment 1 was retained as the preferred alternative.

B.1.4 Diversion and Unwatering Scheme

The cost of diversion during construction was based on the scheme presented in Dwg. B-5. This scheme was assumed to be carried out in two stages as follows:

Stage 1

Approximately half of the right channel is blocked off by a combination of fill cofferdams and timber cribs. The fill cofferams are constructed by end-dumping from the right bank and when completed timber cribs are lowered from these into position into the river bed parallel to the flow. A temporary bridge is then constructed from the end of the upstream cofferdam to the island. Flow meanwhile passes through the remaining part of the west channel and the open east channel.

A second temporary bridge is required on the east channel to allow access to the power canal and powerhouse area during construction.

Stoplog slots are left in extensions to the piers in the concrete overflow dam at the upstream and downstream ends to enable the crest structure to be completed at a later date.

Stage II

The temporary fill cofferdams are removed from the one side of the west channel when construction of the piers have been completed and cofferdams then placed on the open side. The piers and base of the spillway are completed behind the cofferdams after which the cofferdams will be removed. The east channel is closed off completely by construction of a second cofferdam to allow the overflow dam to be completed while the flow passes through the partially completed spillway on the west channel. The crest of the overflow dam on the west channel is completed bay by bay behind stoplogs after completion of the works in the east channel.

B.1.5 Unit Type and Selection

Studies were focused mainly on S-type units and Pit turbines at the prefeasibility stage. Enquiries made to manufacturers indicated that costs of Kaplan units would be too high to be competitive for the head and flow available.

The unit types were compared over a range of heads and it was found that the cost difference (including both civil and mechanical costs) were not significant at the lower heads. However, at the higher heads the Pit turbine had a significant cost advantage.

Multiple unit arrangements were also investigated and it was found that the single Pit turbine had a significant cost advantage over two Pit turbines and three S-type units.

B.1.6 Energy Estimates

Since the Bay d'Espoir Regulation Study was still in progress during the prefeasibility study, the energy estimates were computed, based on the assumption that the development was in

isolation from the rest of the Bay d'Espoir system. Reconstituted monthly flows were taken from a regulation study carried out in 1985 by Acres as part of the Upper Salmon Feasibility Study.

B.1.6.1 Optimization

Four different dam heights, corresponding to full supply levels of 186.5, 188.65, 190.65 and 192.65 were considered. Construction costs in 1987 dollars were developed for each of the heights. The total length of construction was assumed to be three years, starting in 1990 with an on-power date in late 1993. The total construction cost was distributed over the three year period as follows:

Year 1 - 6.8%
Year 2 - 37.0%
Year 3 - 56.2%

Average annual power productions were computed based on a tailwater elevation of 180.7 m, over a 37 year period from 1950 to 1986 for various heads and installation sizes. An efficiency of 90% was assumed for the turbine and 97% for the generator.

The construction costs for civil, mechanical and electrical miscellaneous costs and contingencies in 1987 dollars were inflated at 5% annually to 1992. The annual O & M cost was estimated as follows:

($ Civil) x 0.25% + ($ Mech + Elect) x 1.35%.

For the purposes of calculating the O & M cost per year, the miscellaneous costs were distributed proportionally amongst the civil, mechanical and electrical costs. The annual O & M costs were inflated with 4.23% for 60 years and the present worth in 1987 dollars of the inflated O & M costs computed.

The annual benefit was estimated from the average annual product based on the rate of $0.5/kWh and the total project cost estimated as the sum of the total construction and O & M costs. The net benefit in 1987 dollars was calculated together with the B/C ratio. The estimated cost per kWh of the Development was also computed.

The above procedure was repeated with a current rate of $0.643/kWh as a check on the sensitivity of the analysis.

B.1.6.2 Results of Analysis

The study showed a dam with an FSL of 192.65 to be the most economic. A similar exercise was carried out using a FSL of 192.65 and varying the turbine flows. The most economic flow was found to be 212 m^3/s, giving a B/C ratio of 1.49 for the energy cost of $0.643/kWh and 1.16 for an energy cost of 0.5/kWh.

The average energy for the optimized scheme was 137 GWh and the firm energy 103 GWh.

B.1.7 Capital Cost

The capital cost of the project was estimated to be $71 million including IDC and escalation. A breakdown of the capital cost estimate is shown on Table B-3 on page B-12.

B.1.8 Conclusions and Recommendations

It was concluded from the prefeasibility study that the Round Pond Development appeared sufficiently attractive to proceed with the feasibility study.

The optimized installed capacity was found to be 20 MW with a turbine flow of 212 m^3/s and a FSL of 192.65.

It was recommended that should the decision be made to proceed with the feasibility study particular emphasis should be placed on the following work:

1) Establishing horizontal control for the Development.

2) Site investigation of the saddle dam areas. For the prefeasibility cost estimate quantities for the saddle dam were based on a 1:50,000 scale map.

3) Optimization of the spillway arrangement based on Hydro's decision to adopt the PMF as the design flood.

4) Further refinement of the layout based on the results of the drilling program.

TABLE B-1

SPILLWAY - COMPARISON TABLE

ALTERNATIVE	DESCRIPTION	COMPARATIVE COST
A	Concrete overflow and rockfill overflow structure	6,900,000
B	Gated Spillway (9 gates - 7.5 m high x 7.0 m wide)	7,200,000

TABLE B-2

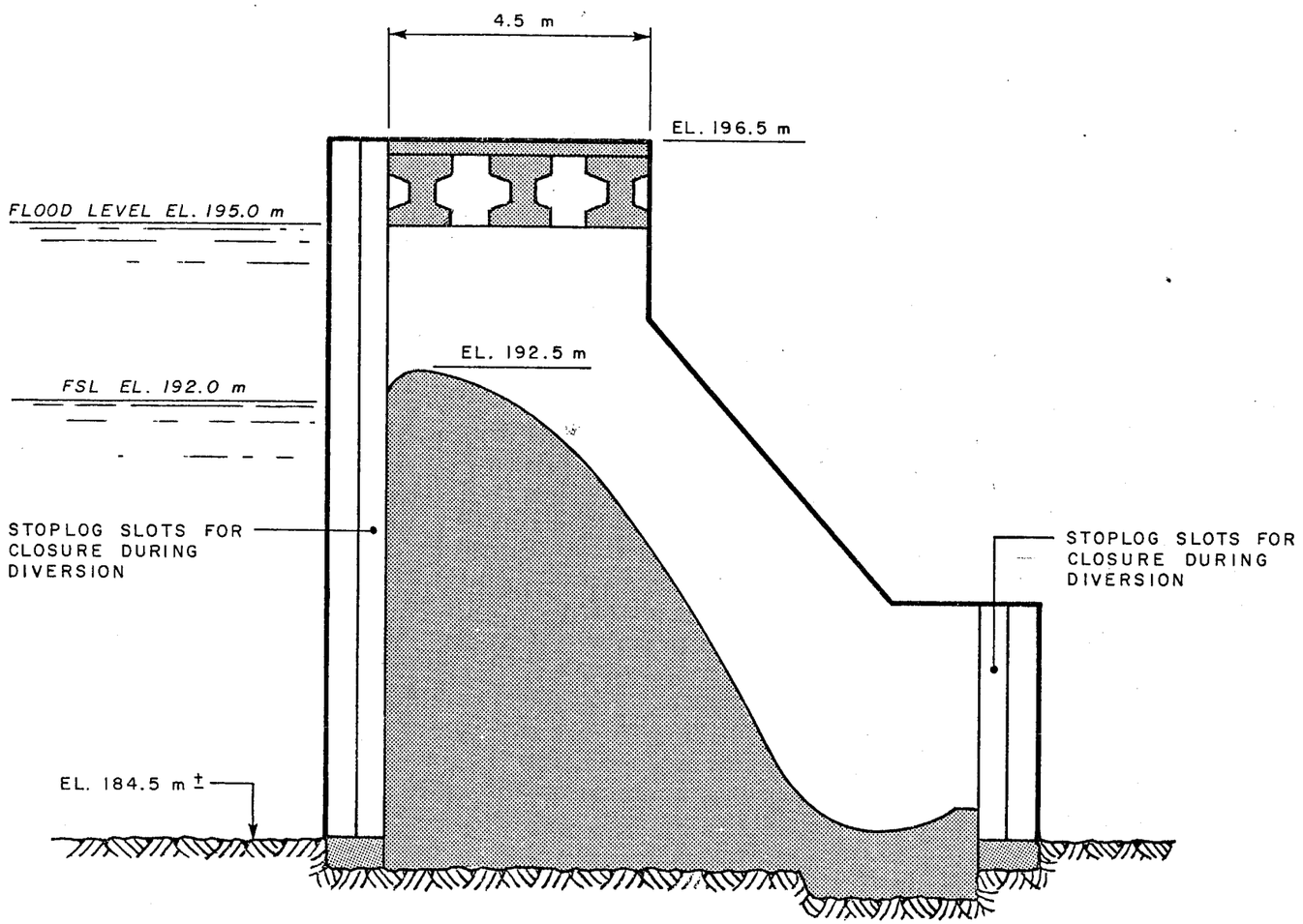
CAUSEWAY ALIGNMENTS - FILL QUANTITIES

ALIGNMENT	EMBANKMENT FILL QUANTITY (m^3)
1	191,000
2	165,553
3	91,000

TABLE B-3

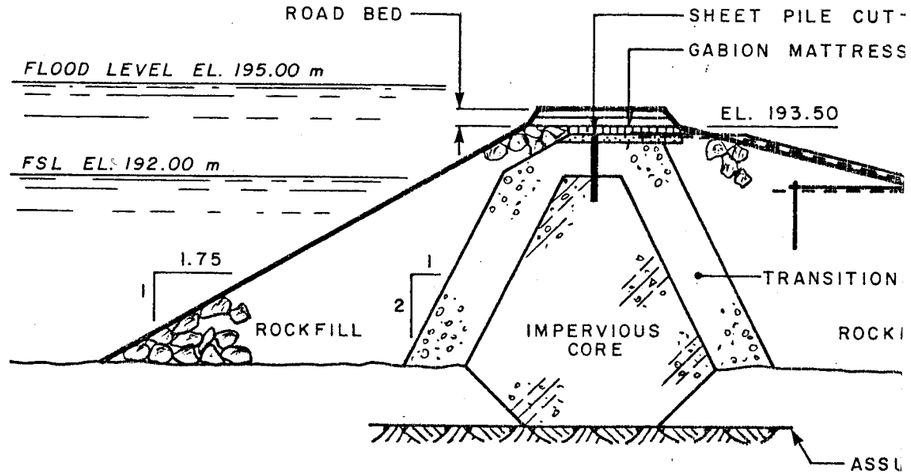
CAPITAL COST ESTIMATES

ITEM	DESCRIPTION	COST $ x 1000
	CIVIL WORKS	
1	Permanent Access Roads	1,653.7
2	Cofferdams and Dewatering	553.8
3	Reservoir Clearing	2,445.0
4	Causeways	2,702.1
5	Spillway	2,587.2
6	Fish Pass	299.4
7	Power Canal	1,732.6
8	Powerhouse Civil Works (incl. Switchyard)	4,509.3
9	Tailrace	130.9
10	Saddle Dams	1,792.3
	Civil Works Sub-Total	18,406.3
	ELECTRO-MECHANICAL WORKS	
12	Hydraulic Gates	4,026.2
13	Generating Plant	10,764.3
14	Miscellaneous Electrical and Mechanical Equipment	2,208.9
	Electro-Mechanical Works Sub-Total	16,999.4
15	Switchyard Equipment	451.0
16	Temporary Facilities	5,378.5
	Construction Cost Total	41,235.2
17	Engineering and Construction Management	6,185.3
18	Owner's Cost	2,061.8
19	Contingencies	6,133.0
	CAPITAL COSTS TOTAL (in January 1988 terms)	55,615.3

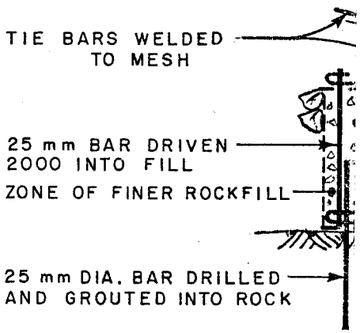


CONCRETE OVERFLOW SPILLWAY

1 : 100



ROCKFIL



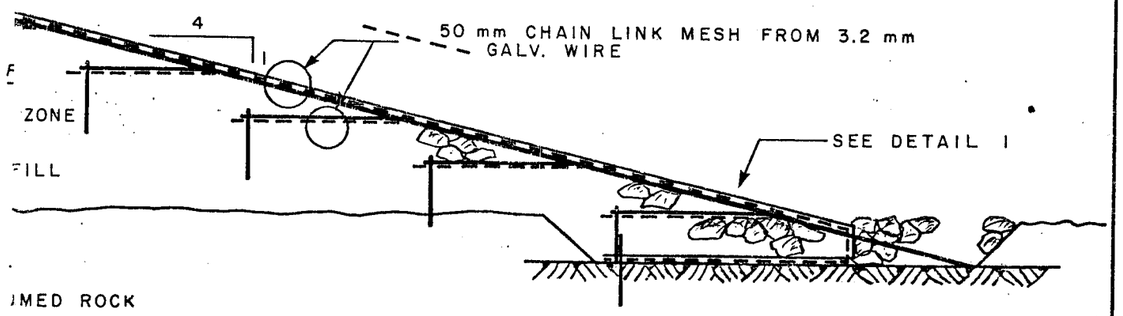
ISSUED : JUNE 88, SEPT. 88

DRAWING B-1

ROUND POND HYDROELECTRIC DEVELOPMENT
FEASIBILITY STUDY
SPILLWAY ALTERNATIVES
-TYPICAL CROSS SECTIONS-

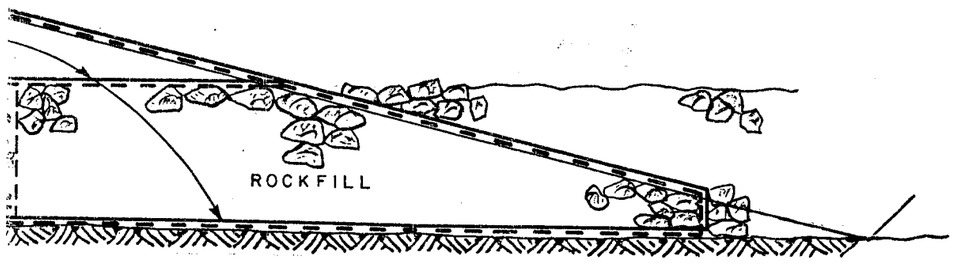
OFF

ON SAND BEDDING



L OVERFLOW SECTION

1:200



DETAIL 1

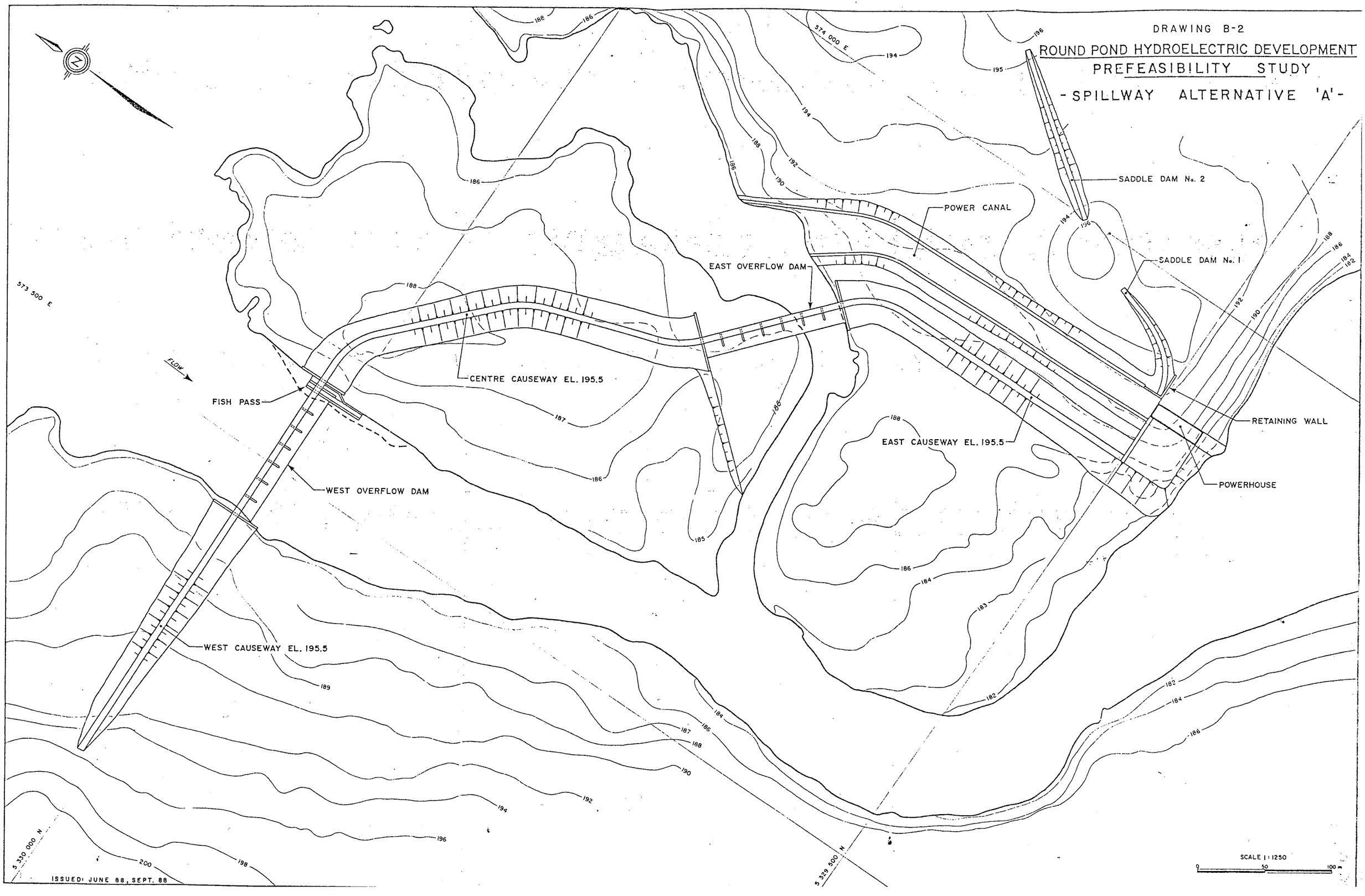
NTS

DRAWING B-2

ROUND POND HYDROELECTRIC DEVELOPMENT

PREFEASIBILITY STUDY

- SPILLWAY ALTERNATIVE 'A' -



SCALE 1 : 1250

0 50 100 m

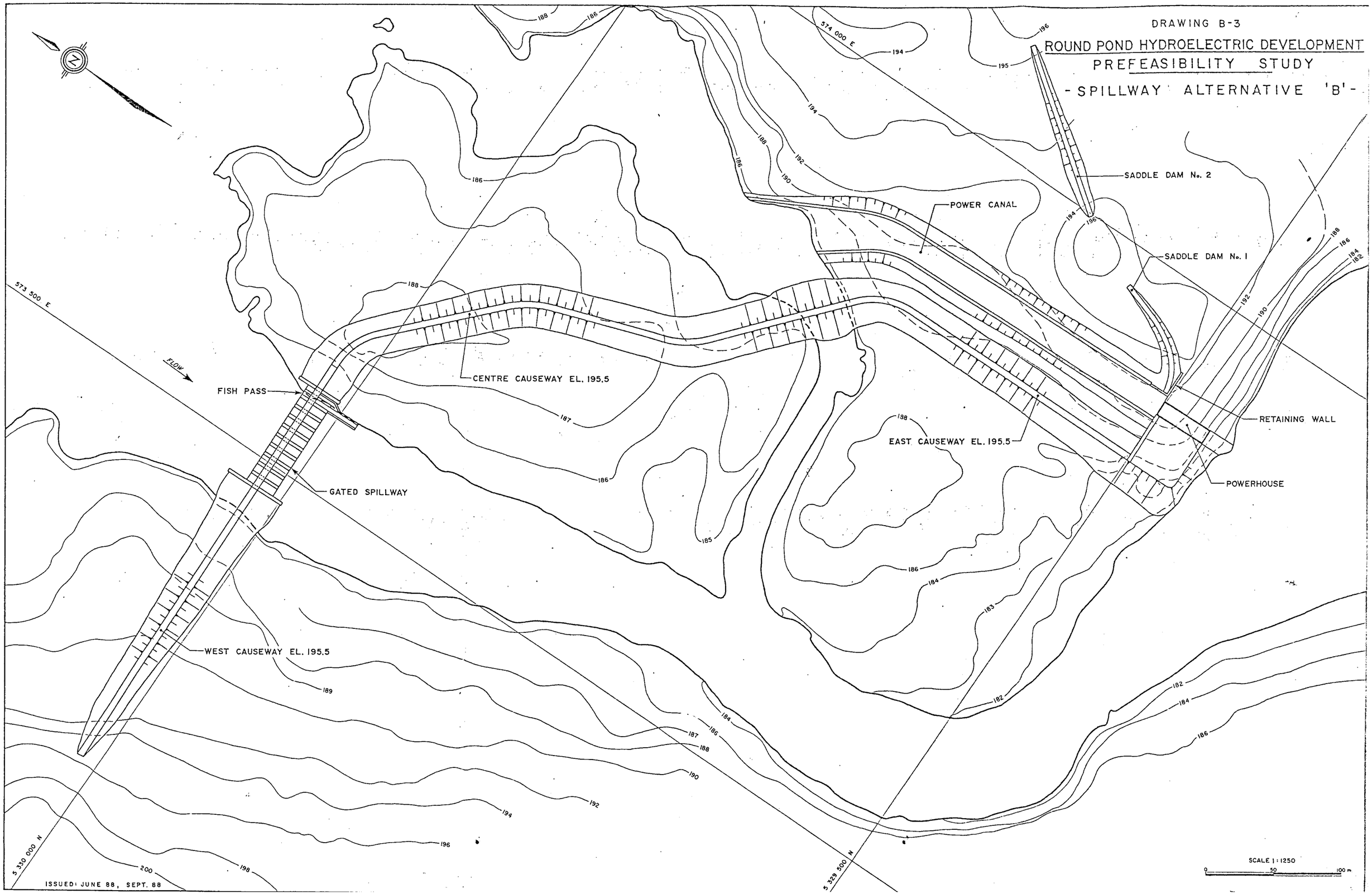
ISSUED: JUNE 88, SEPT. 88

DRAWING B-3

ROUND POND HYDROELECTRIC DEVELOPMENT

PREFEASIBILITY STUDY

- SPILLWAY ALTERNATIVE 'B' -



ISSUED: JUNE 88, SEPT. 88

ROUND POND
W.L. 186.0 m ±

ALIGNMENT No. 1

ALIGNMENT No. 2

ALIGNMENT No. 3

LONG POND RESERVOIR
NTL 180.75 m

STRING BOG

BLDG

5 330 500 N

5 330 000 N

5 329 500 N

574 500 E

575 000 E

ISSUED: JUNE 68, SEPT. 68

DRAWING B-4

ROUND POND HYDROELECTRIC DEVELOPMENT PREFEASIBILITY STUDY

- ALTERNATIVE CAUSEWAY & SADDLE DAM ALIGNMENTS -

ROUND POND
W.L. 186.0 m ±

ROUND POND

SADDLE DAM No. 3
ALIGNMENT 'A'

SADDLE DAM No. 3
ALIGNMENT 'B'

STRING BOG

W.L. 205.3 ± 2

5 331 000 N

•192.3

•203.1

205.5

•197.5

•187.8

W.L. 188.2 ±

192.2

192.6

190.6

194.7

188.9

1903

189.9

189.2

•189.4

190

200

•189.0

W.L. 190.1

191.5

186.7

187.3

185

199.5

210

192

W.L. 199.2 ±

191.4

W.L. 186.8

189.1

198.2

•204.6

192.6

185.2

W.L. 188.8

189.7

190.7

575 500 E

576 000 E

576 500 E

0 100 200 m
(1:2 500)

DRAWING B-5

ROUND POND HYDROELECTRIC DEVELOPMEN
PREFEASIBILITY STUDY
DIVERSION SCHEME

THIS PLAN SHOWS
STAGE I DIVERSION
LEFT CHANNEL OPEN

FLOW

STAGE II COFFERDAM

EL. 187.0

TEMPORARY BRIDGE

DUMPED IMPERVIOUS
& ROCK GROIN

TIMBER CRIB SUPPORT

TIMBER CRIB

FILL COFFERDAM

EL. 187.0

EL. 186.0

E 399 800

E 400 000

E 400 200

N 600 300

N 600 400

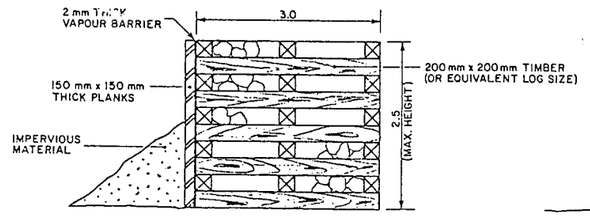
N 600 500

N 600 600

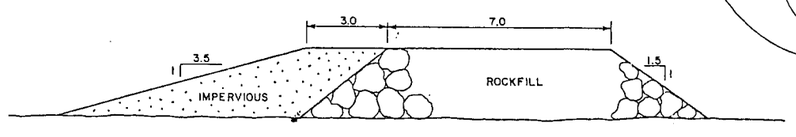
N 600 700

N 600 800

N 600 900



A - A
SECTION THROUGH CRIB



B - B
SECTION THROUGH COFFERDAM

SCALE 1 : 1250

0 50 100 m